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Plain and Reinforced Concrete Arches*

Report of Committee 312

CHARLES S. WHITNEY, CHAIRMAN

SYNOPSIS

This report contains a summary of the work done by Committee 312 since the publication of its progress report in 1932, and presents for discussion a suggested specification for reinforced concrete arch design. Further investigation has been made of the effects of shrinkage and plastic flow. Measurements on large arches at Pittsburgh over a period of years are presented and discussed along with the results of other investigations. Recommendations are made as to the calculations of moments and thrusts due to loading and volume changes. A new method is proposed for the design of the arch rib based on ultimate strength formulas and a new approach to the question of factor of safety for members subjected to direct load and flexure.

I—INTRODUCTION

COMMITTEE 312 submitted a progress report in 1932 (1) relating to the limitations of the theory of elasticity as applied to concrete arches, and the effect of plastic flow, shrinkage and temperature variations. This second report has been delayed pending the completion of other research which was necessary before satisfactory recommendations could be made for arch design. This research related particularly to the effect of plastic flow and shrinkage and to improvements in the method of giving effect to the thrusts and moments in proportioning the arch sections. Sufficient progress appears to have been made in that direction to warrant the submission of a final report presenting a rational method of arch design.

A series of measurements has been made on a number of large arches in Pittsburgh extending over a period of several years to determine the magnitude of shrinkage, plastic flow, and temperature

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movements. A similar investigation but more in detail has been made on the Arlington Memorial Bridge.(2) Further laboratory investigations have been made on shrinkage and plastic flow(3,4,5,6,7,8) and the comprehensive and very important work of the Special Committee on Concrete and Reinforced Concrete Arches of the American Society of Civil Engineers was brought to a close with the submission of a final report in 1935.(9)

These investigations have served to confirm the fundamental principles expressed in the progress report(1) regarding the effect of plastic flow, shrinkage, and temperature changes. But they have also proven the futility of attempting to predict quantitatively the effect of the induced volume changes on unit stresses. This has lead to the recommendation in this report of a new method of design, abandoning the use of unit stress calculations.

It is proposed that thrusts and moments be calculated by the customary methods of the elastic theory. The sections of the arch are then to be proportioned so that these thrusts and moments do not exceed the useful strength of the sections as determined by ultimate strength calculations. The ultimate strength is determined by simple formulas based on the cylinder strength of the concrete and the yield point strength of the reinforcing steel. The useful strength is a proportion of the ultimate strength depending on the desirable factor of safety and the amount of cracking permissible under extreme conditions.

This method is much more realistic than the calculation of working stresses which are not even approximated under actual conditions and it leads to a much more definite and consistent factor of safety through the full range of eccentricities. It will result in considerable economy in material required in the arches in many cases because it recognizes the real strength of the concrete section under bending and direct stress. Also the calculations are much simpler than they are in the case of the standard straight-line-no-tension theory.

This report discusses under the following sections the determination of thrusts and moments, the design of the sections, and closes with a suggested design specification.

II—DETERMINATION OF THRUSTS AND MOMENTS

1. Dead and Live Loads

The reactions of a concrete arch rib and the thrusts and moments at various sections produced by live load can be calculated satisfactorily by application of the theory of elasticity using the transformed moment of inertia of the reinforced sections. This is based

on the customary assumptions that stress is proportional to strain, that plane sections remain plane, and that the concrete is uncracked so that the full moment of inertia is used in the calculations. It has been proven that accidental variations in the modulus of elasticity of the concrete or a considerable amount of cracking do not materially effect the elastic action of the rib.(9)

The reactions, moments, and thrusts due to dead load can also be calculated in the same way as those due to live load. Plastic flow under dead load has the effect of increasing dead load rib shortening but also provides sufficient stress relief as explained in the previous report so that the stresses in the concrete are no greater than they would be if it were purely elastic. The dead-load steel stresses are increased by plastic flow of the concrete, but this increment is not important because it has no effect on ultimate strength as it would be eliminated by the large strains occurring before failure. Dead load rib-shortening thrusts and moments can therefore be calculated as though the concrete were elastic. This is based on the assumption that the arch axis will be designed to follow as closely as possible the pressure line of dead loads in order to reduce to a minimum the distortion of the rib due to plastic flow under dead load eccentricity.

Plastic flow can also be neglected in the case of live loads.

It should be noted that the methods of calculating moments and thrusts are suggested here to be used with the method of design proposed in this report, without consideration as to whether they may or may not be appropriate for use in determining unit stresses. They are considered here entirely from the point of view of their effect on the useful strength of the rib and not on the unit stresses.

The Special Committee of the American Society of Civil Engineers has investigated the action of skewed arches, continuous arches, combinations of arches with deck superstructure and with high piers. Special methods of analysis of such structures is so thoroughly treated in its report (9) that they will not be discussed here.

Tables and diagrams for use in determining dead and live load effects on symmetrical and unsymmetrical arches are available (10,11) as well as for elastic constants to be used for calculating effects of shrinkage, temperature, and elastic supports such as continuous spans or yielding abutments. If the arches are proportioned as directed, these give accurate solutions in accordance with the elastic theory and save a great amount of labor. Live load effects should be calculated with the use of influence lines in order to determine the maximum thrusts and moments due to critical positions of the loads.

Consideration should be given to construction loads in case heavy equipment or materials or other construction conditions may have a more critical effect than the regular live loads for which the arch is designed.

2. Temperature Changes

In addition to the data summarized in the progress report, the following observations have been made on temperature effects on arches.

In the case of one of the arches of the Arlington Memorial Bridge,(2) the initial temperature of the concrete when pouring was begun was about 75° F. After about 25 hours it reached a maximum of 143° F. The evolution of heat of setting continued after the maximum temperature was reached but the temperature of the concrete dropped gradually and reached air temperature in about 15 days. This measurement was made at the springing where the concrete was about four feet ten inches thick. At the crown where the barrel was 27 in. thick, the maximum temperature varied between 90° and 100° F and the concrete reached air temperature in about 8 days after pouring.

This gives an indication of about how long the pouring of transverse keys in the arch should be delayed after the main body has been poured in order to eliminate the effect of temperature rise due to setting. Some of this effect will be eliminated by plastic flow, but there is reason to believe that the rapid setting of the present standard portland cement has effected a reduction of the plastic flow at such early ages that the heat of setting may cause cracking in heavy arches unless precautions are taken.

It was further observed that after the heat of setting was dissipated the mean temperature of the arch closely approximated the mean air temperature for the five previous days. During a period of 27 months the maximum variation in mean concrete temperature was 61° F while the maximum range of air temperature was 99° F. The concrete was made with Potomac River sand and gravel and had a coefficient of thermal expansion of 0.0000065 per degree F. as determined from prisms and measurements of crown deflection. The stiffness of the spandrels and deck reduced the vertical movement of the crown about 13 per cent as compared with the free arch and increased the bending in the arch near the springing. After 25 months the concrete showed a compressive strength of 5400 p.s.i. and a secant modulus of elasticity of 4,200,000 p.s.i.

The concrete of the Rogue River Bridge arches (18) had a strength of about 5000 p.s.i. at 76 to 98 days and an average modulus of elasticity of 4,630,000 p.s.i. The coefficient of thermal expansion was 0.0000053

per degree F. It was made of one part of cement and 2.75 parts of aggregate consisting of high grade local sand and gravel derived from basaltic rock. The water-cement ratio varied from 0.593 to 0.653 and the cement content was 8.48 sacks per cu. yd. Temperatures at the axis of the rib increased to a maximum of about 120° to 140° F 30 hours after pouring with the air temperature at 60° F and dropped to air temperature in about 100 hours after pouring. The rib averaged a little over three feet thick and was six feet wide.

In the case of the Pittsburgh tests to be described later in this report, the movement of the arch crowns appeared to correspond to a thermal coefficient of 0.000006 for the gravel concrete and to 0.000004 for the limestone concrete of the Westinghouse Bridge when the mean air temperature of the previous 5 days was used except in the case of the very thick arches of the Jacks Run Bridge where the 8 day average seemed to give closer results.

It is recommended that for the purpose of design the range of temperature of the arch rib be assumed as 60 per cent of the range between extreme air temperatures at the locality. It may be considered that keying and plastic flow will eliminate the effect of heat of hardening on the rib and that plastic flow will permit the arch to adjust itself to mean temperature so that it can be calculated for equal rise and fall.

The coefficient of thermal expansion may be assumed as 0.0000055 per degree F. unless information can be obtained as to the actual materials and coefficient. It may be 0.0000065 for gravel concrete or as low as 0.000004 per degree F. for limestone concrete. A special investigation of the coefficient should be made for cases where the temperature effects are very important.

A value of 4,000,000 p.s.i. is recommended for the modulus of elasticity of the concrete and the calculation of temperature thrusts is to be made as though the rib were elastic and uncracked. The stresses due to difference in thermal coefficients of the steel and concrete may be neglected because they do not effect the useful strength of the arch. A single value of modulus of elasticity is recommended for this purpose because it seems best after consideration of many factors involved. Because of the effect of age, speed of loading and other factors it is difficult to determine the actual modulus of elasticity. Too low a value should not be used because of the importance of temperature thrusts on abutment and piers. It does not appear that a higher value need be used because the temperature effects must be relieved somewhat either by plastic flow ([12]) or a reduction in modulus

of elasticity which has been observed under repeated loading.[13] This maximum value also seems justified by the nature of the temperature thrust which is immediately relieved by any cracking or yielding of the rib under high stresses so that it cannot cause failure but is eliminated by the strains occuring long before ultimate load is reached. The purpose of provision for temperature effects is principally to prevent undesirable cracking of the rib in service and this is quite important in the case of arches with a low rise-span ratio.

3. Shrinkage and Plastic Flow

Since 1932, tests on arches and large specimens have indicated that the actual shrinkage and plastic flow movements are smaller than the figures used in the calculations in the progress report. The principles expressed regarding their effects on stresses appear to be correct but a simplified treatment in design of ribs is recommended. The effect of plastic flow on rib-shortening due to load has already been discussed and it is suggested that the effect of shrinkage be reduced to an equivalent elastic shortening and be included with the temperature effect calculation, neglecting the additional effect which it has on the steel stresses.

It is recognized that the drying of large members of concrete is very slow and that the occasional wetting or immersion in fog of arches exposed to the weather will permanently prevent the loss of much moisture from a greater part of the section.

An elaborate investigation of shrinkage was made in connection with the Rogue River Bridge.[18] An 8- by 16-in. cylinder was cast with a telemeter at the center and protected from direct wetting by rain but otherwise exposed to the effects of atmospheric humidity. The total shrinkage over a period of 2425 hours was 0.0007 in. per in. A full size section of the reinforced rib six feet long cast beside the bridge and exposed to the same weather conditions showed a shrinkage of only 0.00015 in the same period. During this period there was no rain but occasionally heavy fogs enveloped the bridge and produced a measurable swelling of the concrete.

The average shrinkage measured at a large number of points on the spans during 100 days after pouring was 0.00011 and the individual measurements varied from 0.00002 to 0.00027 because of varying conditions. Some were wet longer than others and some were exposed to direct sunlight. There was practically no rain during this period and most of the time a stiff breeze was blowing.

During the construction of the Arlington Memorial Bridge in Washington and for a period of about two years after the pouring

of the arches in 1929, accurate measurements were made to determine the effects of temperature, shrinkage and plastic flow.(2) The concrete was made of one part portland cement, 1¾ parts sand, and 3½ parts of gravel. It showed a 28 day strength of 3900 p.s.i. The dimensions of Arch 7 on which the observations were made are given in Table 1.

Fig. 1 shows the unit strains in the arch due to shrinkage and plastic flow as nearly as they can be deduced from the movement of the crown of the arch. Corrections have been made for temperature movements, distortion due to deck construction, and deck restraint. These influences introduce some uncertainty and unfortunately no attempt was made to isolate the shrinkage effect by direct shrinkage observations but the results do give a good indication of the magnitude of the combined effect of shrinkage and flow. The upward trend during the winter months is probably due to increased humidity of the air.

The total unit strain at the end of the first two years was about 0.00036. If the shrinkage strain during this period was about 0.00015 as would appear reasonable from other tests, the balance due to plastic flow would be 0.00021 or about 0.00000064 per lb. per sq. in. This unit strain is calculated from the full dead load stress which was not on the arch for the full period so the flow factor must have been somewhat higher.

In the calculation in the progress report of the effect of flow in relieving the bending moments due to shrinkage the unit shrinkage was taken as 0.0003 and the flow factor as 0.000001. This showed a

TABLE 1—DIMENSIONS OF BRIDGES INVESTIGATED FOR LONG TIME DEFLECTION

Description of Bridge	Arch No.	Span (Ft.)	Rise (Ft.)	Width of Rib (Ft.)	Depth of Section		Area of Long. Steel (Sq. in.)
					Crown (Ft.)	Spring. (Ft.)	
George Westinghouse	2	255	98	14	3	6	43.7
	3	411.4	158	14	5	10	53.0
	4	255	98	14	3	6	43.7
Ohio River Blvd. over Spruce Run	1	150	41.75	10	3	5	37.4
	2	150	41.75	10	3	5	37.4
	3	150	41.75	10	3	5	37.4
Ohio River Blvd. at S. Fremont St.	1	118	41	10	3	5	37.4
Ohio River Blvd. over Jack's Run	1	400	90	10	8	14.25	62.5
Arlington Memorial over Potomac River	7	177.33	24.33	87.33	2.25	5.9	3.00 per ft.

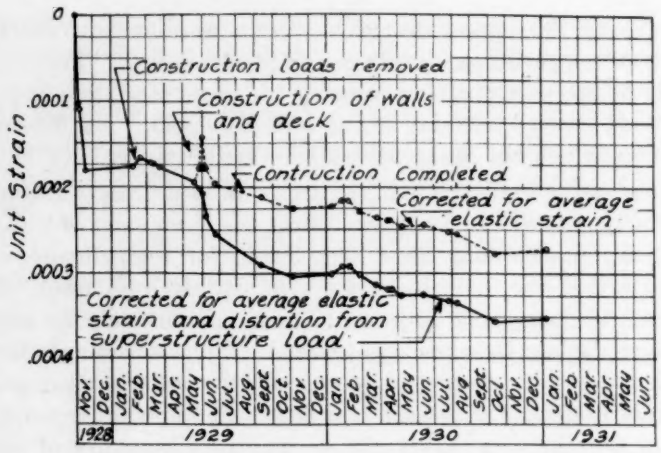
0
.0001
.0002
.0003
.0004
Unit Strain
Construction loads removed
Construction of walls and deck
Contruction Completed
Corrected for average elastic strain
Corrected for average elastic strain and distortion from superstructure load
Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun. Jul. Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May Jun.
1928 1929 1930 1931

FIG. 1—UNIT STRAIN CAUSED BY SHRINKAGE AND PLASTIC FLOW ARCH NO. 7, ARLINGTON MEMORIAL BRIDGE, WASHINGTON, D. C.

release of about 79 per cent of the shrinkage stress, making the shrinkage effect equal to a drop of about 12° F. in temperature. It appears reasonable to assume now that the actual shrinkage may be about 0.00015 and that the flow factor will be sufficient to release at least 60 per cent of the shrinkage stress. This would reduce the shrinkage effect to that of a unit shortening of 0.00006 which is equivalent to 10° F with a thermal coefficient of 0.000006 or 15° F. if the thermal coefficient is only 0.000004.

Fig. 2 and 3 give the crown deflection measured at the request of Committee 312 on four bridges constructed by Allegheny County* under the supervision of Vernon R. Covell, Chief Engineer of Bridges, and George Richardson in charge of design.[19] Unfortunately it was not possible to make arrangements to start the observations as soon as the arches were constructed. Dimensions are given in Table 1.

The concrete was 1:2:4 mix with an average strength of 2600 p.s.i. Crushed limestone was used for the George Westinghouse Bridge and gravel for the others.

The dotted lines indicate the actual observed elevations (average of opposite curbs) at the center of spans and over the piers. The full lines connect the points which have been corrected for temperature variations as noted in the previous section.

*The measurements were made by Department of Highways, Bridges and Tunnels, Allegheny County, through the courtesy of Charles C. McGovern, Chairman of the Board of County Commissioners and the cooperation of Thomas O. Hasley, and J. F. Laboon, Directors of the Department, Park H. Martin, Chief Maintenance Engineer, and C. K. Harvey and Wm. McClurg Donley, Maintenance Engineers.

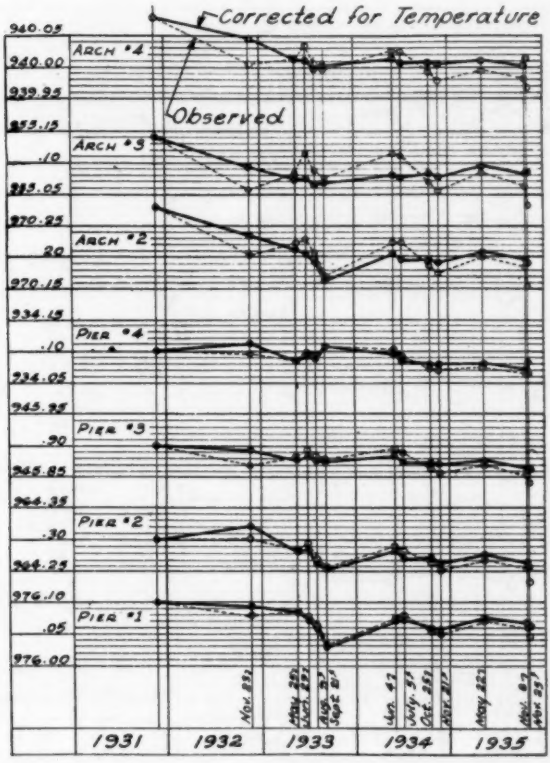

FIG. 2—DEFLECTION OF GEORGE WESTINGHOUSE BRIDGE, PITTSBURGH, PA.

The first elevations shown for the George Westinghouse Bridge are the plan elevations at which the curbs were built.

According to the best information available, the bridge was actually constructed to these levels but this is not certain. A further uncertainty regarding total crown movement of the arches is due to the fact that the movement caused by distortion of the ribs, when the columns and deck were constructed, was not definitely determined. Because these observations were started so late, it is possible to draw only general conclusions but these are important. There was no indication of appreciable shrinkage or flow after the second year of life of the arches although the crowns apparently respond to temperature and probably moisture changes. In general, the levels at the tops of the piers (which in some cases are very high and would not be much effected by plastic flow) indicated a total shrinkage of about 0.00015.

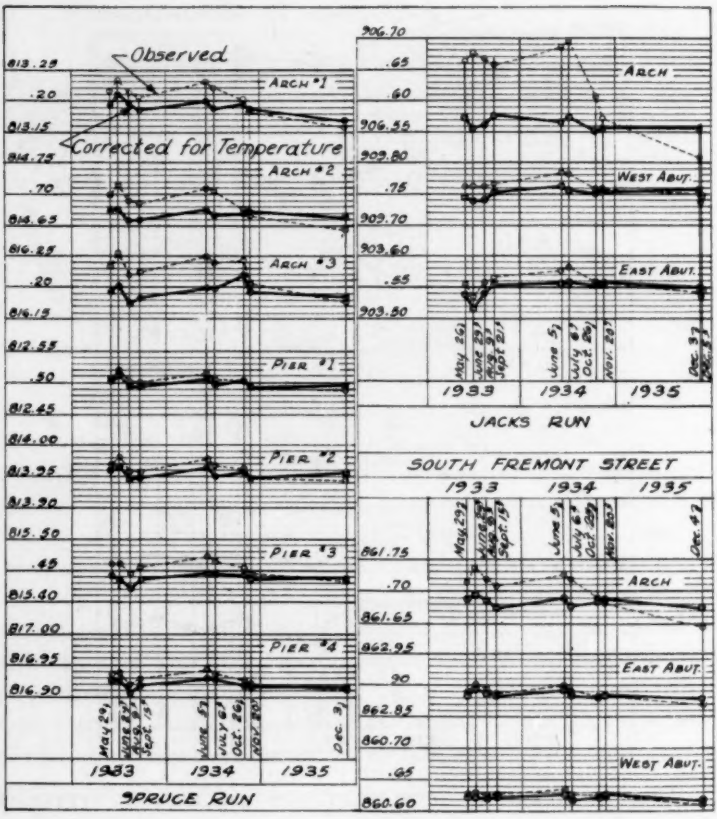

FIG. 3—DEFLECTION OF OHIO RIVER BOULEVARD BRIDGES, PITTSBURGH, PA.

One other interesting fact was noted in the case of these arches as well as the Arlington Memorial Bridge. When the superstructure was built on the completed arches, the crowns rose as the deck was placed over the haunches and came back practically to the original position when the center portion was completed. This must mean that the distortion of the ring due to placing the columns or cross walls and the deck forced the crown up in these cases enough to compensate for the effect of rib shortening. Of course, this action is effected by many factors and might not occur in other cases.

In view of these data it is recommended that the effect of shrinkage be considered in design as equivalent to the effect of a drop in temperature of 15° F. For calculating the shrinkage deflection of arches

for camber it could be assumed to produce a rib-shortening of about 0.00015 in. per in.

In figuring temperature, shrinkage, and dead load rib shortening thrust, it is recommended that a modulus of elasticity of 4,000,000 p.s.i. be used for the concrete. For calculating dead load deflections, about one third to one fourth of this value should be used to allow for plastic flow. The method of taking into account the effect of volume changes here suggested is considered to be perfectly safe because it is known that the rate of flow increases at higher stresses in such a manner as to prevent dangerous overstress if reasonable allowances are made.[12]

III—PROPORTIONING OF ARCH RIBS

Because of the large number of indeterminate factors involved, it has become apparent that it is impossible to predict the unit stresses in an arch rib with any degree of accuracy. Furthermore, the nominal stresses calculated by the best methods available do not bear any consistent relation to the actual ultimate or useful strengths of the rib. Consequently, a new method of calculating the strength of an arch rib has been derived which is based on a recognition of the plastic qualities of the concrete. The principles underlying this method have been explained elsewhere [14,15,16] and need not be treated in detail here but the derivation of formulas for arch ribs will be outlined. These formulas give the ultimate strength of the rib as controlled by compressive strength for the smaller eccentricities and by the tensile strength of the steel for larger eccentricities and cover the full range between pure bending and axial compression in a consistent manner.

It has been shown that the flexural strength of a rectangular concrete member as controlled by the compressive strength of the concrete and that of the compression steel is given by the formula[16]

$$M_c = \tfrac{1}{3}\, bd^2 f'_c + d'A'_s f_s$$

in which b = the width of the section.

d = depth from the compression face to the tension steel.

d' = the distance from compression steel to tension steel.

f'_c = standard cylinder strength of concrete.

A'_s = area of compression steel.

f_s = yield point stress in the steel or the stress which produces a total unit strain of 0.004.

This resisting moment is the same whether the bending is produced by a couple, alone or by a couple combined with a thrust, and it can be used to give the strength of an arch rib. Therefore when there is sufficient tension steel to prevent a tension failure, the strength of the arch rib can be derived as follows, by taking moments (see Fig. 4).

$$M_e = P\left(e+d-\frac{t}{2}\right) = \tfrac{1}{3}\, bd^2f'_c + d'A'_sf_s$$

from which, if $d = \dfrac{t+d'}{2}$

$$P = \frac{2A'_sf_s}{\dfrac{2e}{d'}+1} + \frac{btf'_c}{\dfrac{3te}{d^2}+\dfrac{6dt-3t^2}{2d^2}}$$

This formula gives the correct value when the eccentricity of load is greater than the eccentricity of the resisting forces in the compression side of the section. Because of its derivation it has no theoretical meaning for smaller eccentricities but it can be adjusted to the smaller range by making P approach the proper value for an axially loaded column as e approaches zero. The first term giving the steel strength needs no adjustment but it is known that the concrete strength is $0.85\ btf'_c$ for axial load so the second term becomes

$$\frac{btf'_c}{\dfrac{3te}{d^2}+1.178}$$

and with this change, the last equation becomes:

$$P = \frac{2A'_sf_s}{\dfrac{2e}{d'}+1} + \frac{btf'_c}{\dfrac{3te}{d^2}+1.178} \qquad \text{(Eq. 1)}$$

which gives the strength of the rectangular arch rib as controlled by compression. This gives values which agree satisfactorily with tests as will be shown later.

As indicated by Fig. 4, ([16]) it is assumed that at failure the compression resistance can be represented by an equivalent rectangular stress block of average intensity equal to $0.85f'_c$. It should be emphasized that this method is not based on an assumption of a rectangular distribution of stress although an equivalent rectangular stress block is used. It is based on the actual stress-strain curve of concrete

cylinders in compression. It is also assumed that the stress in the compression steel will be equal to the yield point stress of the steel in the case of compression failure. This is consistent with the standard method for axially loaded columns.

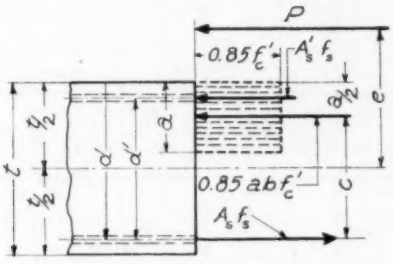

FIG. 4

The formulas for the strength of the rib as controlled by ultimate tensile strength of the steel for the larger eccentricities will now be derived. If the rib is over-reinforced on the compression side so that the compression steel is sufficient to take the total compression force without help from the concrete, the value of the ultimate load can be expressed by taking moments, and

$$P = A_s f_s \frac{2d'}{2e - d'} \qquad \text{(Eq. 2)}$$

When there is not sufficient compression steel to take all the compression force, it may be assumed for practical purposes that both the tension and compression steel will be stressed to the yield point at ultimate load, and the balance of the compression load on the section acts on the compression side of the concrete over an equivalent depth equal to a. This total compression on the concrete is then $0.85\,abf'_c$ and it follows that

$$P + A_s f_s = A'_s f_s + 0.85abf'_c$$
$$\text{or } 0.85abf'_c = P + (A_s - A'_s)f_s$$

$$\text{and } a = \frac{P}{0.85bf'_c} + \left(A_s - A'_s\right)\frac{m}{b}$$

$$\text{in which, } m = \frac{f_s}{0.85f'_c}$$

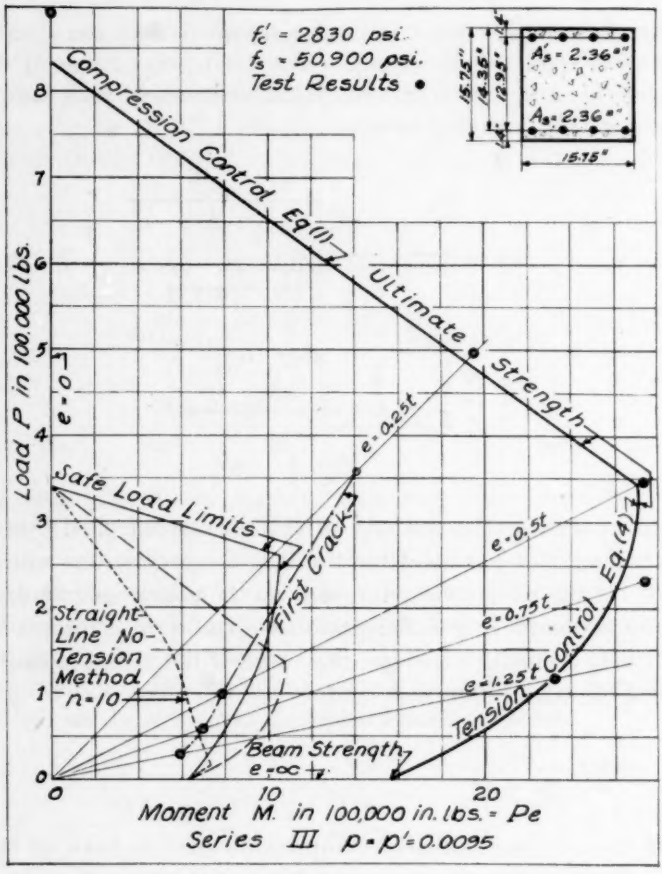

FIG. 5

Then taking moments about the tension steel

$$M = P\left(e + \frac{d'}{2}\right) = d'A'_s f_s + 0.85 f'_c ab\left(\frac{d'}{2} + \frac{t}{2} + \frac{a}{2}\right)$$

Substituting the previous values and solving for P, we find:

$$P = 0.85tbf'_c \left\{ \sqrt{\left[\frac{e}{t} - 0.5 + (p - p')m\right]^2 + \frac{2d'}{t} p'm} \right.$$

$$\left. + (p - p')m\left[\frac{d'}{t} + 1 - (p - p')m\right] - \left[\frac{e}{t} - 0.5 + (p - p')m\right] \right\} \text{(Eq. 3)}$$

For the symmetrically reinforced rib when $p = p'$, this reduces to

$$P = 0.85tbf'_c \left\{ \sqrt{\left(\frac{e}{t} - 0.5\right)^2 + \frac{d'}{t} p_t m} - \left(\frac{e}{t} - 0.5\right) \right\} \quad \text{(Eq. 4)}$$

in which, $p_t = p + p' = \dfrac{A_s + A'_s}{bt}$

When there is no compressive steel, $p' = O$ and

$$P = 0.85tbf'_c \left\{ \sqrt{\left(\frac{e}{t} - 0.5 + pm\right)^2 + \left(\frac{d'}{t} + 1 - pm\right)pm} - \left(\frac{e}{t} - 0.5 + pm\right) \right\} \quad \text{(Eq. 5)}$$

On account of the assumption that the steel on the side of the rib away from the load is in tension equations (2), (3) (4) and (5) have no significance when e is very small but that case is covered by Eq. (1).

Table 2 and Fig. 5, 6, and 7 show the agreement between these formulas and the results of tests made by Bach and Graf ([17]) on 49 columns 16 in. sq. The concrete had a 12 inch cube strength of 225 kg. per sq. cm. which is equivalent to a 6 by 12 inch cylinder strength of 2830 p.s.i. The steel had a yield point of 53700 p.s.i. and 50900 p.s.i. with p equal to 0.005 and 0.0095 and p' equal zero, 0.005 and 0.0095. The tests therefore covered a range which is particularly significant for arch design. The agreement between formulas and tests is remarkably close, with the actual strengths slightly larger in all cases.

The equations also show very close agreement with the strength of knee frames tested by Richart & Olson and by Hayden ([16]) in which the eccentricity was greater.

The loads at which the first crack appeared are also plotted in Fig. 5, 6, and 7.

It should be noted that in these figures the load P is plotted as the ordinate with M or Pe (instead of e/t) as the abscissa. The radial lines are lines of equal values of e/t. The Y-axis represents the case of axial load ($e = O$) and the X-axis the case of pure flexure ($e = \infty$ and $P = O$).

The upper or compression control line plotted from Equation (1) is a straight line connecting the point on the Y-axis representing the strength of the member when axially loaded with the point on the X-axis corresponding to the pure flexural strength of the member

TABLE 2—COMPARISON OF ACTUAL AND CALCULATED ULTIMATE LOADS FOR COLUMNS TESTED UNDER ECCENTRIC LOADS BY BACH AND GRAF [17]

(See Fig. 5, 6, 7 & 8)

Series	p	p'	Eccentricity e/t	No. of Tests	Actual Load (lbs.)		Calculated Ultimate Load (lbs.)	Equation No.	Actual Ultimate Load Divided by Calculated Load
					Ultimate	First Crack			
I	0.005	0	0	3	618,000		596,000	1	1.04
			0.50	3	204,700	75,700	203,000	5	1.01
			0.75	3	132,700	44,800	123,300	5	1.07
			1.25	3	66,000	23,500	62,500	5	1.07
II	0.005	0.005	0	3	735,000		729,000	1	1.01
			0.25	2	447,000	293,000	423,000	1	1.05
			0.50	3	273,000	77,800	257,000	4	1.06
			0.75	2	153,400	45,200	148,000	4	1.03
			1.25	2	71,300	22,300	68,500	4	1.04
III	0.0095	0.0095	0	3	892,000		835,000	1	1.07
			0.25	2	496,000	358,000	486,000	1	1.02
			0.50	3	347,000	99,000	343,000	1 and 4	1.01
			0.75	2	231,500	58,900	219,000	4	1.05
			1.25	3	118,000	27,700	116,400	4	1.01
IV	0	0	0	3	609,000		597,000	1	1.02
			0.25	3	300,000	208,000	299,000	6	1.00
			0.375	3	180,500	91,900	149,000	6	1.21
			0.50	3	52,800	52,500	0	6	

as controlled by the strength of the compression side assuming sufficient tension steel to develop it.

The lower or tension control line plotted from Equation (3), (4) or (5), as the case may be, is a curve which reaches the X-axis at the point representing the beam strength or the flexural strength of the member as controlled by the strength of the tension steel. Any point lying in the area between the two control lines and the X and Y axes represents a combination of load and moment which the member will carry without failure but if the point lies to the right of the control lines, failure will result.

It is particularly important to note that when the bending moment is greater than the beam strength of the member, failure may be caused by a reduction of the direct load without a change in the bending moment. For at least the lower part of the tension control curve, the ultimate moment increases with the increase in direct load.

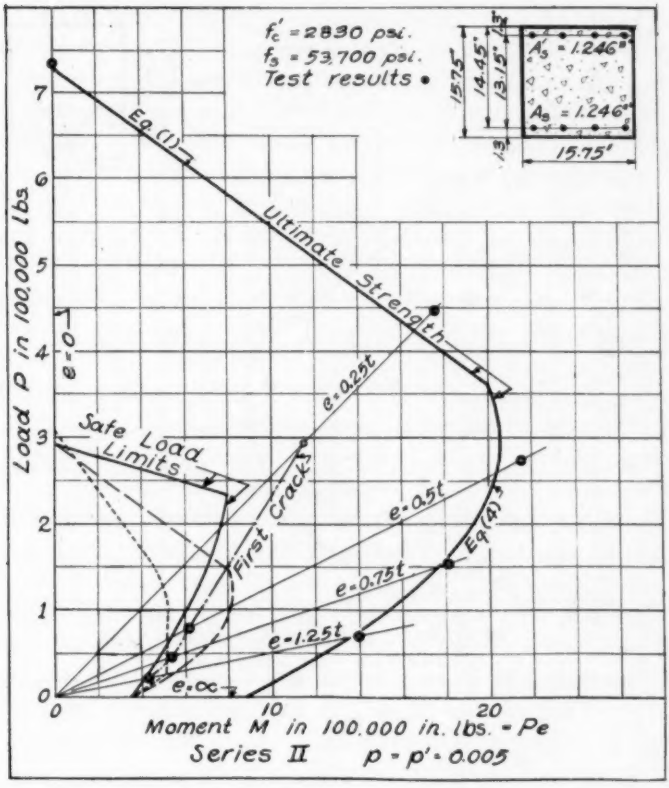

FIG. 6

It is therefore not safe to assume as is customarily done that in case of overload the eccentricity remains constant and that both direct load and moment are multiplied by the same factor. It is apparent that an arch rib or any other member subjected to both direct load and bending cannot be designed on the same basis as a column or beam under pure compression or bending alone. Instead of using a simple factor of safety which assumes a fixed relation between load and moment, it is necessary to consider the effect of any possible combination of load and moment.

The direct thrust in an arch rib is largely due to dead load except in small spans. It can therefore be calculated with reasonable accuracy and is not subject to great increase or decrease. On the other hand, bending is produced largely by live loads and volume changes which are much more uncertain than dead load. It therefore appears

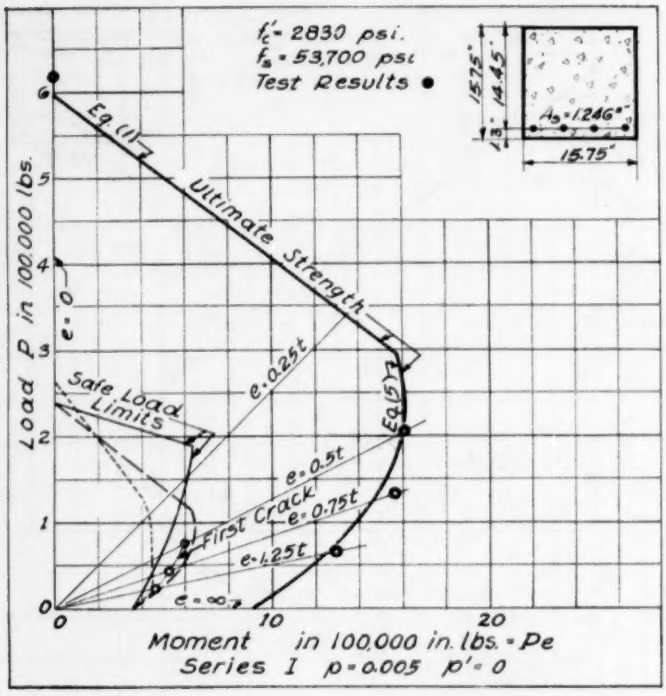

FIG. 7

best to design the rib to resist a multiple of the moment combined with the actual calculated thrust. This should be conservative because there is no possibility of a material reduction in the thrust, and what possible small increase there might be would only serve to increase the resistance to bending. If the same multiple were used for both moment and thrust, the result would be a reduction in the factor of safety in the most probable case of an increase in moment without an increase in thrust.

It is proposed that the arch be designed so that the effect of total thrust due to dead load, live load, rib shortening, shrinkage and temperature combined with two and one half times the total moment due to the same causes shall not exceed the ultimate strength of the rib. This would actually provide a factor of safety of considerably more than two and one half against flexural failure because of the nature of the moments due to volume changes and due to redistribution of moments in the rib before failure. It has been demonstrated experimentally that moments due to volume changes do not materially

effect the maximum load-carrying capacity of the arch rib.[12] It is suggested, however, that the maximum moment under working conditions (including effects of dead and live loads, shrinkage, plastic flow and temperature variations) should be kept down to 40 per cent of the ultimate strength in order to prevent objectionable cracking in service. 40 per cent of the ultimate seems a fair estimate of the useful strength of the rib in bending. In the case of a beam it is customary to allow a steel stress equal to 50 per cent of the yield point which is actually 50 per cent of the ultimate strength of the beam. It will be noted that while the first pronounced crack in a beam may occur at about 25 to 30 per cent of the ultimate, Fig. 5, 6 and 7 show that it occurs at higher and higher percentages as the eccentricity decreases. In Series I and II with about one half of one per cent of tensile steel, the first crack occurred at moments higher than 40 per cent of the ultimate (measured along the horizontal lines representing constant thrust, Fig. 6 and 7) while in Series III with about one per cent of tensile steel it was so close to 40 per cent that no permanent damage could be done to the rib if the extreme combination of all moments did go that high occasionally.

In order to prevent overloading a rib with a thrust of small eccentricity, it is necessary to establish a compression control, and the rib should be designed so that its ultimate strength is sufficient to withstand the total bending moment due to all causes combined with a proper multiple of the thrust. This factor is designed to keep the load within the useful strength and provide safety against accidental variations and inaccuracies. A factor of about $2\frac{1}{2}$ seems appropriate but it is suggested that because of the different nature of dead and live loads, a factor of two be used for dead load thrust and three for live load thrust and shrinkage and temperature effects. Since the fatigue limit of concrete in compression under repeated loads is over 50 per cent of the ultimate,[13] this would keep the actual loads well within the useful strength. It would provide a better factor of safety for light arches where the live load is more important and at the same time permit the concrete to be used more efficiently for long heavy spans where the dead load is greater in proportion.

The light solid lines in Fig. 5, 6, and 7 indicate the limits of the useful strengths of the three ribs tested. The straight line for the lower values of e have 40 per cent of the ordinate of the compression strength curve and the curved line for larger values of e have 40 per cent of the abscissa of the tension strength curve. The rib is safe for any combination of thrust and moment lying in the area between these two curves and the X and Y axes.

For comparison, the dash lines show the useful strength area for the rib on the assumption that thrust and moment would be increased proportionately (resulting in constant value of e) and a factor of safety of 2½ is used. It is seen that this would result in higher allowable value of moment for the larger eccentricities where cracking might become serious and in lower values of thrust and moment for the lower eccentricities where there is no danger of cracking. This does not appear as satisfactory as the method proposed.

The dotted lines show the strength of the ribs as calculated by the standard straight-line-no-tension method using the actual maximum thrust and moments, a unit concrete stress of 40 per cent of the cylinder strength and a steel stress of 40 per cent of the yield point with $n = 10$. This results in a great reduction in the allowable loads on the rib and an excessive and inconsistent factor of safety for the middle range of eccentricities. It does not properly recognize the true compressive strength of the rib in the range of eccentricities which is really critical for most arches. For the very high eccentricities, it lies near the dash line and actually exceeds the values which would be permitted by the proposed method. The present standard method gives results which are entirely inconsistent with the true safe strength of the rib. The proposed method will result in thinner, more economical ribs with a consequent reduction in dead load and volume change effects.

For instance, the rib of Series III would be used for a thrust of 116,300 lb. with a moment of 912,000 in. lb. ($e = t/2$) by the proposed method while the standard unit stress method would require an increase in depth of 23 per cent with a proportional increase in steel to carry the same load.

For the design of plain concrete sections Equation (5) may be used, and with $p = O$, it becomes simply

$$P = 0.85tbf'_c \left(1 - \frac{2e}{t}\right) \quad \text{(Eq. 6)}$$

Fig. 8 shows the agreement between this equation and the series of tests by Bach and Graf on unreinforced members. It will be noted that the first crack appeared at a higher proportional load than in the case of the reinforced columns. With $e = 0.5t$, the first crack and ultimate failure occurred at almost the same load.

It has been customary in the past to design large, heavy ribs in the same manner as vousoir arches keeping the pressure lines within the middle third of the section (e not exceeding $t/6$) to prevent tension entering the concrete. This would provide the calculated factor of

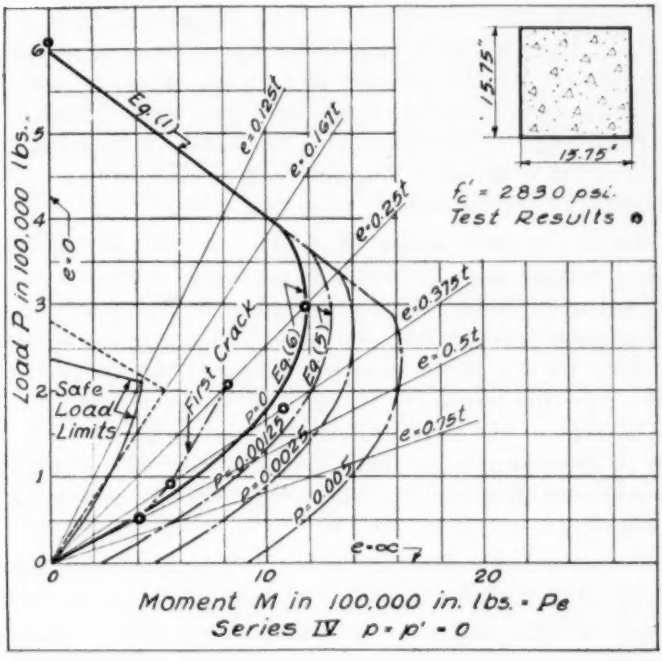

FIG. 8

safety only if thrust and moment vary proportionately in case of over-load but it does not provide for the additional eccentricity occurring if the moment were increased without increase in the thrust, which is a very possible condition. By using (as suggested for reinforced arches) a safe load limit of 40 per cent of the maximum moment given by Eq. (6) combined with the actual thrust, a better and more conservative control will be established than merely limiting the maximum eccentricity to $t/6$. This safe load limit is shown in Fig. 8 compared with the dotted line representing the ordinary method with e less than $t/6$, and the unit stress due to direct load and bending equal to $0.4f'_c$.

The use of plain concrete arch ribs is not recommended.

The design formulas presented are based on a plastic distribution of the stress in the concrete without considering the tension. The possibility of cracks from over-load or shrinkage or some accidental cause makes the recognition of tension in the concrete inadvisable except in making calculations of elastic properties for the determination of thrusts and moments. Because of the large number of variable factors in arch design it does not appear practical at present to draft

rules as to the best proportions of ribs for special cases. But the simplicity of these formulas and the tables and diagrams[10] available make it possible for the designer to test the effects of variations in width, crown and springing thicknesses, rise, span, amount of reinforcement, strength of materials, etc., to determine the most economical solution. It is probable that few such studies have actually been made because of the inadequacy of design methods.

The use of the same factor of safety for both the steel and the concrete presupposes, of course, well controlled concrete work. The value of concrete strength f'_c, to be used in design should correspond to the minimum value of field tests, not the average, so as to eliminate the necessity for any factor of safety to cover uncertainty of concrete strength. On this basis, the strength of the concrete should be as reliable as that of the steel. Concrete in a bridge arch has the advantage of increasing in strength after 28 days due to exposure to atmospheric humidity while this increase cannot be depended on in a heated building.

For the present, the stresses to be used in design should not be higher than 5000 p.s.i. for f'_c and 50000 p.s.i. for the steel yield point. Dependable 3000 pound concrete can be made with ordinary care, and that value could be used for general design with 40000 p.s.i. for f_s corresponding to the standard specification for intermediate grade.

Tests reported by W. M. Wilson[9] indicate that if an arch rib is not subjected to lateral forces, its strength is not effected by its slenderness so long as the ratio of the unsupported length of the arch axis to the width of the rib does not exceed 30. For more slender arches provision should be made to prevent buckling by providing lateral support or reducing the loads.

Since very high stresses may be produced in the steel, it should be effectively spliced and tied at proper intervals to prevent buckling. Twisted bars should not be used for longitudinal reinforcing because they are not fully effective in compression and have a splitting effect on the concrete.

IV—SUGGESTED SPECIFICATIONS FOR CONCRETE ARCH DESIGN*

1. Arch Axis.

The shape of the arch axis shall follow the equilibrium polygon for dead loads as nearly as practicable in order to reduce dead load distortion.

*These proposed specifications are presented with the three supporting parts (I, II, III) of the report as information and are not at this time offered with a view to adoption as an Institute Standard.

2. Moments and Thrusts.

The moments and thrusts acting on the arch shall be calculated by means of the elastic theory based on the following assumptions:

(a) A plane section before flexure remains plane after bending.

(b) The concrete is uncracked and elastic and has the same modulus of elasticity at all sections and all stresses.

3. Modulus of Elasticity.

For the purpose of computing the reactions, the modulus of elasticity of the concrete shall be assumed to be 4,000,000 p.s.i. In determining dead load deflections one third to one fourth of that value shall be used.

4. Live Load Effect.

The live load thrusts and moments shall be calculated with the loads placed in positions giving maximum effects. Critical construction loads if any shall be considered in design.

5. Temperature Changes.

Reactions due to temperature changes shall be calculated for a rise and fall equal to 30 per cent (a total range of 60 per cent) of the difference between maximum and minimum recorded temperature at the locality. The coefficient of thermal expansion shall be taken as 0.0000055 per degree Fahrenheit unless tests show that some other value should be used.

6. Shrinkage.

Reactions due to shrinkage of the concrete shall be equal to the effect of a 15° F drop in temperature. In calculating the combined effect of shrinkage and temperature changes, this 15° F for shrinkage effect shall be added to the temperature drop but shall not be deducted from the temperature rise because of the possibility that shrinkage may at times be offset by moisture conditions.

7. Strength of the Rib.

The ultimate strength of the rib as given by Eq. (1) shall not be exceeded at any section by the total bending moment due to the combined effects of dead and live loads, temperature change and shrinkage in combination with twice the dead load thrust plus three times the thrust due to other causes.

The ultimate strength of the rib as given by Eq. (4) (use Eq. 3 or Eq. 5 if the reinforcement is not symmetrical) shall not be exceeded by the total thrust due to the combined effects of dead and live loads,

temperature change and shrinkage in combination with 2½ times the total bending moment due to all causes.

The concrete strength, f'_c, to be used in the equations, shall not exceed the strength of 6″ by 12″ control cylinders tested at 28 days and shall not exceed 5,000 p.s.i. The steel strength f_s, shall not exceed the yield point of the steel in tension and shall not exceed 50,000 p.s.i.

8. Width of Rib.

The ratio of the unsupported length of the rib axis to the width of the rib shall not exceed 30 unless provision is made to prevent buckling.

9. Abutments.

The abutments or piers shall be designed to avoid uneven or excessive pressures on the foundations which might cause undesirable movement or yielding.

10. Continuous Action.

Continuity with the deck, piers or adjacent spans and the location of expansion joints shall be given suitable consideration as the particular conditions may require.

11. Reinforcement.

Concrete shall not be depended upon to resist tension and sufficient steel shall be used at all locations to resist tensile forces.

Preferably, longitudinal bars shall be placed symmetrically near the extrados and intrados with cross bars spaced not farther apart than 30 diameters of the main bars and securely wired at all intersections. Ties or stirrups connecting through the arch not less than ¼ in. or ¼ the diameter of main bars shall be provided at least at alternate main bars spaced longitudinally not more than 30 diameters of the main bars. If such ties are not provided at all main bars they shall be staggered so that they are spaced not more than 60 diameters along each main bar.

Cross bars shall be designed to resist the bending from spandrel walls.

The splices in the main longitudinal bars shall be welded to develop their full strength or shall be lapped not less than the following lengths:

Deformed bars:

Structural or intermediate grade	40 diameters
Hard Grade	50 "

Plain Bars:

Structural or intermediate grade	50 "
Hard Grade	62.5 "

Splices shall be located if possible away from the locations of maximum stress and if this is not practical semi-circular hooks shall be provided at the bar ends.

Twisted bars shall not be used for main arch reinforcement.

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Discussion, to close in April, 1941* JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1, 1941.

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Discussion of the report of Committee 312:

"Plain and Reinforced Concrete Arches"*

CHARLES S. WHITNEY, CHAIRMAN

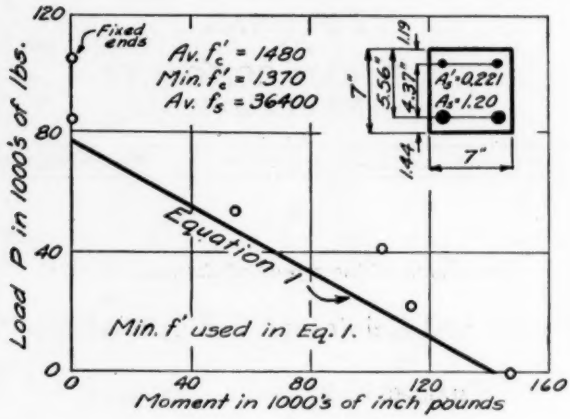
BY GEORGE C. ERNST AND CHARLES S. WHITNEY

BY PROFESSOR ERNST†

Since the proposed formulas in Part III, upon which Specification 7 is based, are relatively new‡, it appears appropriate to direct attention to the column tests reported in 1938 by the Department of Scientific and Industrial Research (Great Britain).** Although the British tests were conducted on considerably smaller columns than those of Bach and Graf, they covered the entire range from e = 0 to pure bending and also included one group which a much higher strength concrete (av. f'_c = 6060 p.s.i. for 4 in. cube). Steel percentages, in most cases, were typical of arch design. Several tests were also made with varying eccentricities, leading to the conclusion that a 50 per cent reduction in the initial value of the eccentricity might be taken as the extent of redistribution in a framework utilizing normal concrete mixtures.

Fig. A to C provide a comparison between actual test results and the formulas submitted by Committee 312. The 4-in. cube strengths were converted to cylinder strengths by the ratio of cube column factor (0.65 in British report) to cylinder column factor of 0.85. Table A shows the essential data in the same manner as Table 2 of the Committee Report. The curves were constructed by use of the minimum concrete strength and average steel yield point in the group considered.

*JOURNAL, Amer. Concrete Inst., Sept. 1940; *Proceedings* Vol. 37, p. 1.
†Civil Engineering Dept., Syracuse University, Syracuse, N. Y.; formerly of University of Maryland.
‡First proposed in 1937 in the paper cited as Reference 14 at the close of the Committee Report.
**"The Strength and Deformation of Reinforced Concrete Columns Under Combined Direct Stress and Bending." F. G. Thomas, Technical Paper No. 23 on Building Construction.



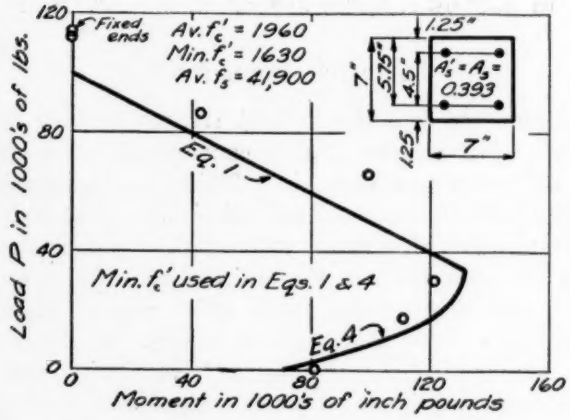

Fixed ends
Av. f'_c = 1960
Min. f'_c = 1630
Av. f_s = 41,900
Eq. 1
Min. f'_c used in Eqs. 1 & 4
Eq. 4
Load P in 1000's of lbs.
Moment in 1000's of inch pounds
$A'_s = A_s$ = 0.393

FIG. A—SERIES I, COMPRESSIVE CRITICAL (BRITISH TESTS)

FIG. B—SERIES II, $p = p' = 0.0080$ (BRITISH TESTS)

The tests show good agreement with the proposed formulas excepting for the higher strength concrete. A decrease in the ratio of plain column strength to cube strength for the higher quality concrete was noted in the British tests and was no doubt a contributing factor but does not fully correct for the discrepancy shown. The use of nominal or minimum steel yield point stress would shift all curves slightly toward the origin but would not correct for the relative difference between groups. It might be desirable, therefore, to place a limit on f'_c at a value somewhat lower than the 5000 psi. suggested on p. 22 and p. 24 of the committee report, at least until tests have been conducted in sufficient number to show definitely the action of the higher

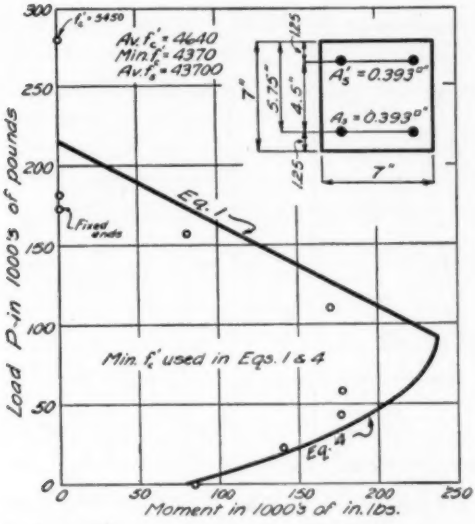


FIG. C—SERIES III, $p = p' = 0.0080$ (BRITISH TESTS)

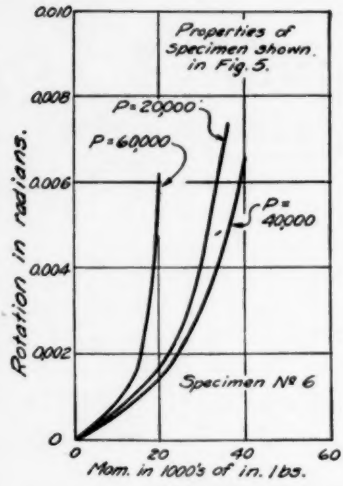
strength concrete when subjected to bending and direct stress. Results from tests made at the University of Maryland* with varying eccentricities lend supporting evidence to the British tests since concrete

*"Tests of Considere Hinges Under Direct Stress, Bending, and Shear," JOURNAL Amer. Concrete Inst., Sept. 1939, *Proceedings* Vol. 35. These tests were conducted as a portion of an investigation of Considere articulations but the central portion of each specimen, although small, was essentially a very short column. The spiral was inactive at the loads at which rotation was applied.

TABLE A—COMPARISON OF ACTUAL AND CALCULATED ULTIMATE LOADS FOR COLUMNS TESTED UNDER ECCENTRIC LOADS BY F. G. THOMAS, DEPT. OF SCIENTIFIC AND INDUSTRIAL RESEARCH, GREAT BRITAIN

Series	P	e'	$t/$	No. of tests	Actual ultimate load	Calculated[3] ultimate load	Eq. no.	Actual ult. load / Calc. ult. load
I	0.0245	0.0045	0	2	85,300	77,200	1	1.10
			0[1]	4	105,600	77,200	1	1.37
	Av. = 1480		0.143	2	54,450	51,100	1	1.06
	f'_c. Min. = 1370		0.357	2	41,700	34,100	1	1.22
	f_s. Av. = 45600		0.715	2	22,900	21,900	1	1.05
II	0.0080	0.0080	0	4	113,800	100,900	1	1.13
			0[1]	2	113,900	100,900	1	1.13
	Av. = 1960		0.071	2	86,500	81,400	1	1.06
	f'_c. Min. = 1630		0.214	2	66,100	59,000	1	1.12
	f_s. Av. = 41900		0.571	2	30,300	33,400	4	0.91
			0.857	2	18,500	20,700	4	0.90
III	0.0080	0.0080	0	3	183,200	216,400	1	0.85
			0	2	280,500	216,400	1	1.31[2]
			0[1]	2	174,200	216,400	1	0.81
	Av. = 4640		0.071	2	157,800	172,300	1	0.92
	f'_c. Min. = 4370		0.214	2	112,400	123,100	1	0.91
	f_s. Av. = 43700		0.429	2	58,600	77,700	4	0.75
			0.571	3	43,700	51,900	4	0.84
			0.857	3	23,100	25,800	4	0.90

[1] Tested with fixed ends. All other columns tested with knife-edge end bearings.
[2] The cylinder strength of these columns exceeded the average by 17.5 per cent.
[3] Min f'_c and av. f. used in each group.



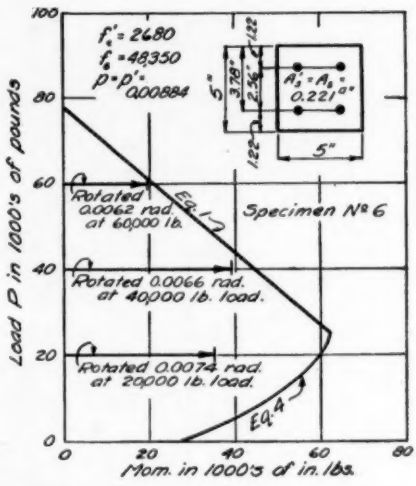


FIG. D—DEVELOPMENT OF MOMENT AT CONSTANT THRUSTS

FIG. E—EXTENT OF MOMENT APPLICATION IN UNIVERSITY OF MARYLAND TESTS

having a strength of 4490 psi. did not show any marked difference in moment development as compared with a lower strength concrete (2680 psi.) at equivalent rotations.

The Maryland tests provide a slightly different picture of the action of reinforced concrete subjected to combined bending and direct stress although entirely consistent with the Committee's interpretation of tests made with fixed eccentricities. Fig. D and E illustrate the results obtained from one of the specimens by holding various loads constant while moments were superposed by means of special equipment. The axial design load based on the A. C. I. Code would be 24000 lb. The maximum moments and resulting rotations superposed upon the 20,000-, 40,000-, and 60,000-lb. thrusts were purposely severe. No visible damage was evident for the combined loading at 20,000-lb. thrust, but crushing of the outer fibers on the high compression side was quite noticeable at 40,000 lb., and at the 60,000-lb. combination definite spalling occurred. Permanent axial compressive deformations produced at the 20,000-lb. load with the highest rotation were not disconcerting but those at 40,000 and 60,000 were definitely becoming critical at their highest rotations. It also appears likely from Fig. D and E that this specimen would not have developed a moment as large as indicated by Equation 4 at 20,000 lb. although at 40,000 and 60,000 the agreement with Equation 1 is good. All comparable specimens

showed the same results. This was true in the British tests (Fig. B) on small cross sections in the tensile control territory. It is possible that the size of specimen aids in producing the entire condition shown in Fig. C but such a conclusion is hardly justified at this time since the number of tests on reasonably large square columns (with tied reinforcement in two opposite faces only) covering the entire range from $e = 0$ to pure bending appears to be extremely meager. The writer has a distinct preference for the proposed formulas over those resulting from the "straight-line-no-tension" theory but additional tests will be needed to establish complete confidence in them for the higher strength concretes when subjected to bending and direct stress.

BY MR. WHITNEY*

The Committee appreciates Professor Ernst's interest in the report.

While the writer believes with Professor Ernst that more tests are needed, he does not agree that the tests already made indicate that the equations are not correct for concrete strengths up to 5000 psi. In his analysis of the Thomas tests, Professor Ernst used in the formulas the minimum concrete strength and the average steel strength for each series. He therefore, did not get a true check on the accuracy of the formula as he would if correct values were used in each case. If the individual strengths are used for each case, the comparison of actual to computed strengths of the columns is given in Table B following the form of Professor Ernst's Table A.

The writer believes that these results are probably caused by two conditions.

*Chairman Com. 312.

TABLE B—COMPARISON OF ACTUAL AND THEORETICAL LOADS FOR THOMAS TESTS

Item No.	Actual Load Divided by Computed Load	Item No.	Actual Load Divided by Computed Load
Series II, Average f'_c = 1960 psi.		Series III, Average f'_c = 4640 psi.	
6	0.919	12	0.778
7*	0.993	13†	1.059
8	0.922	14*	0.787
9	1.007	15	0.845
10	0.827	16	0.875
11	0.905	17	0.754
		18	0.860
Average	0.930	19	0.860
		Average	0.853

*Fixed Ends. †f'_c — 5400 p.s.i.

First, inadequate anchorage of the reinforcing steel caused slipping of the bars and splitting of the concrete ends. Contrary to Professor Ernst's statement, the writer believes this would effect the tests with large eccentricity as well as the others and Table B indicates no consistent trend. There is no doubt but that the efficiency of the specimens was reduced by the inadequate ends.

Second, the control tests were made on 6-in. cubes and the tests on plain concrete columns made at the same time showed the following relations between column and cube strengths:

Series	6 in. Cube Strength	Column Strength Divided by Cube Strength
N E	2460 p.s.i.	0.67
F E	5780 p.s.i.	0.61

This reduction of the ratio of column to cube strength for the stronger concrete might be expected to account for a reduction in the strength of the reinforced columns. This may indicate that the standard 6- by 12-in. cylinder is a better control specimen than the 6-in. cube because the column tests, made at the University of Illinois, showed no such trend. They are reported as follows:*

	Cylinder Strength p.s.i.	Ratio of Plain Column to Cylinder Strength
Air Storage	2515	0.92
	4565	0.79
	5705	0.88
	Average	0.86
Moist Storage	2685	0.71
	4445	0.68
	6510	0.75
	Average	0.71

Even more conclusive evidence that there is no reduction in the relative capacity of members of concrete with strengths up to at least 5000 psi. is furnished by the high strength specimens of the Slater and Lyse tests (Table 1) and the Brandtzaeg tests.† In the range above 5000 psi., evidence is lacking. There is no definite explanation

*"Fourth Progress Report on the Column Tests Made at the University of Illinois" by F. E. Richart and G. C. Staehle, A.C.I. JOURNAL, Jan. 1932; *Proceedings* Vol. 28, p. 296.

†"Der Bruchspannungzustand und der Sicherheitsgrad von rechteckigen Eisenbetonquerschnitten unter Biegung oder Aussermittigem Druck," by Anton Brandtzaeg, Norwegian Technical High School, Trondheim, 1935.

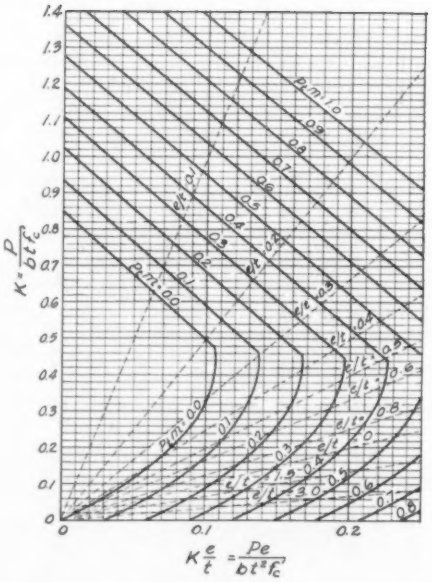


FIG. G—ULTIMATE STRENGTH OF RECTANGULAR SECTION SYMMETRICALLY REINFORCED, $d = 0.85t$

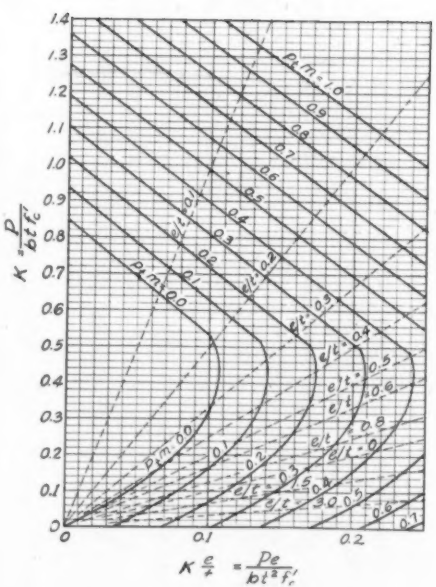


FIG. H—ULTIMATE STRENGTH OF RECTANGULAR SECTION SYMMETRICALLY REINFORCED, $d = 0.9t$

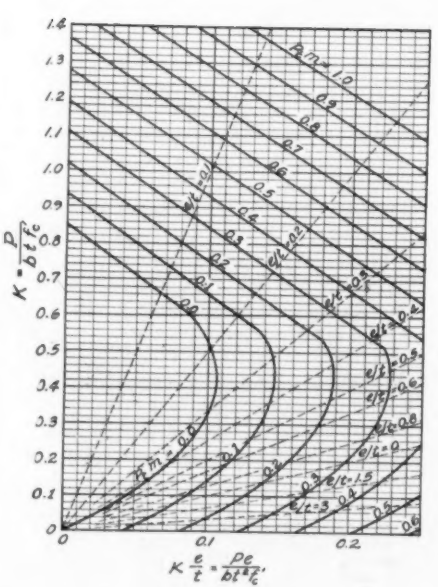


FIG. I—ULTIMATE STRENGTH OF RECTANGULAR SECTION SYMMETRICALLY REINFORCED, $d = 0.95t$

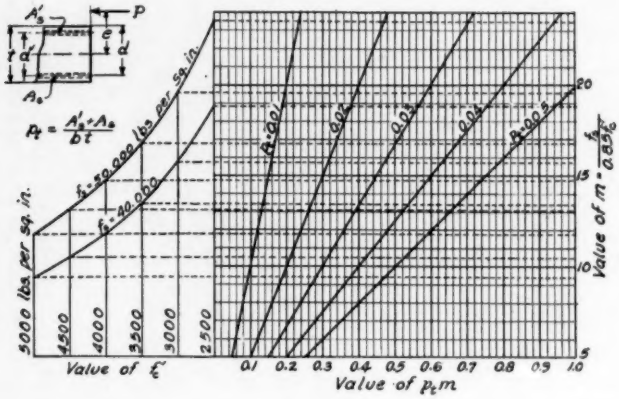


FIG. F

for the lack of strength of the Richart and Olson specimens but it is significant that those under axial load were also deficient.

In order to simplify the application of Eq. (1) and Eq. (4) and to encourage further study, the writer has prepared Fig. F, G, H and I for the direct solution of symmetrically reinforced rectangular sections.

Fig. F gives the value of $p_t m$ for various values of f_s, f'_c, and p_t.

Fig. G gives the ultimate strength of a symmetrically reinforced rectangular section for any value of $p_t m$ and e/t when d is equal to $0.85t$. Fig. H and I give corresponding values when d equals $0.90t$ and $0.95t$ respectively.

The recommendations in this report represent a rather marked change from current practice, and the unusual conditions prevailing since the publication of the report have prevented thorough consideration and wide discussion. The Committee believes that the specifications presented, with whatever modifications that may be decided upon, will result in considerable economy and increased efficiency in concrete arch design, and it therefore hopes that full consideration will be given to its adoption as a standard as soon as conditions permit.

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Tunnel Lining Methods for Concrete Compared*

By Lewis H. Tuthill†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The comparative merits of various methods for lining tunnels with concrete are frequently in question. The question includes consideration of equipment as well as methods. The purpose of this paper is to answer these questions in the light of current experience.

The subjects of leakage, shrinkage, cracking, and contraction joints are treated because they are inseparably involved in the results of placing methods.

1. PRINCIPLE REQUIREMENTS OF VARIOUS TUNNEL LININGS

Tunnels are lined for a number of reasons and are built to serve widely different purposes. These variables largely affect the extent to which the points raised herein concerning lining methods are applicable and economically justified. The most appropriate method and the proper limit of refinement must be determined for each job after its requirements have been considered fully. In this connection it seems desirable to mention the principal tunnel types and what their linings should do and should not do.

Water Supply Tunnels have a considerable size range, are most commonly circular or of horse-shoe shape; more often than not they are longer than other tunnels, and operate either under pressure or at hydraulic grade. Depending on the operating pressure, value of the water, surplus capacity, damage which may be caused by water leaking out, contamination or suits for depletion which may result from water leaking in, and construction costs to prevent leakage, the matter of leakage assumes varying degrees of importance. The extent to which leakage matters has an important bearing on the choice

*Received by the Institute July 8, 1940.
†Engineer, Bureau of Reclamation, Denver, Colorado. (Formerly testing engineer during construction of the Colorado River aqueduct.)

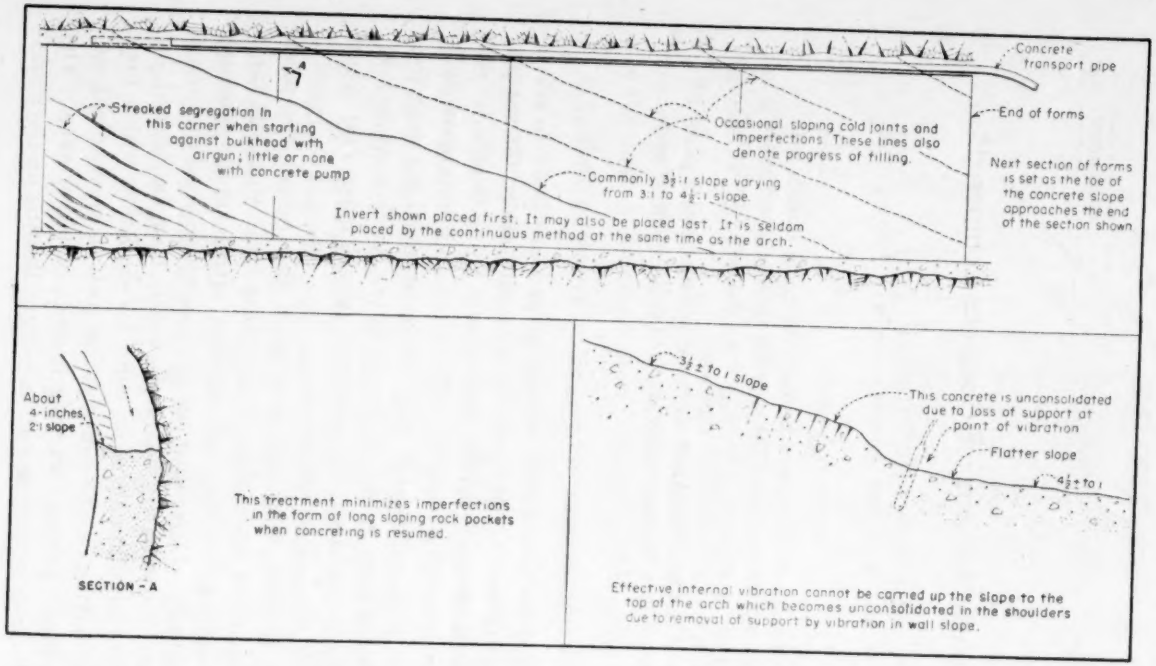
Concrete transport pipe
Streaked segregation in this corner when starting against bulkhead with airgun; little or none with concrete pump
End of forms
Occasional sloping cold joints and imperfections. These lines also denote progress of filling.
Next section of forms is set as the toe of the concrete slope approaches the end of the section shown
Commonly 3½:1 slope varying from 3:1 to 4½:1 slope.
Invert shown placed first. It may also be placed last. It is seldom placed by the continuous method at the same time as the arch.
About 4-inches 2:1 slope
SECTION - A
This treatment minimizes imperfections in the form of long sloping rock pockets when concreting is resumed.
3½ to 1 slope
This concrete is unconsolidated due to loss of support at point of vibration
Flatter slope
4½ to 1
Effective internal vibration cannot be carried up the slope to the top of the arch which becomes unconsolidated in the shoulders due to removal of support by vibration in wall slope.

FIG. 1 (TOP)—TUNNEL LINING BY CONTINUOUS ADVANCING-SLOPE METHOD FROM A VERTICAL BULKHEAD OR FROM A SLOPING COLD JOINT

Principal disadvantage of this method is that all efforts toward compaction, particularly by vibration, are largely dissipated in movement down the slope which unconsolidates the concrete above (see Fig. 3) and makes more forms necessary.

Result is that concrete must be run wet enough largely to compact itself with a minimum of working.

Air gun placing produces least separation when used in connection with the advancing slope method, provided the start is always made on a well grouted slope and not against a vertical construction joint, and the end of the discharge pipe is kept deeply buried in the arch concrete.

FIG. 2 (BOTTOM LEFT)—TREATMENT OF SURFACE OF CONCRETE FOR A SLOPING COLD JOINT

FIG. 3 (BOTTOM RIGHT)—EFFECT OF VIBRATION OF CONCRETE ON A SLOPE IN THE FORM

of lining methods. Linings must not only be strong enough to sustain the section but should be of sufficient surface quality, particularly in the invert, to preserve the initial efficiency of flow throughout years of operation.

Miscellaneous water tunnels for spillway, drainage, diversion, or pen-stock purposes in general require less refinement in lining than the water supply tunnels. They are generally shorter although they may have a similar variety of shapes. Leakage into or out of these tunnels is usually of small importance. Preservation of top-notch efficiency of flow is not a principal requirement although the inverts of spillway and drainage tunnels should in some cases have good resistance to abrasion. An ordinary amount of transverse cracking is not particularly objectionable. Strength to sustain the section must of course be obtained. If "ordinary"construction methods, later discussed, are justifiable anywhere, it is in the lining of these miscellaneous water tunnels.

Highway or railway tunnels are of fairly standard sections and are in general comparatively short. Linings of these tunnels serve not only to maintain the section but also the confidence of the public in their safety. For the latter reason random transverse or other cracking and leakage into the tunnel is to be avoided particularly in highway tunnels. If the ground is dry, only the cracking need be controlled by frequent construction joints; if the ground is wet, water stops at the construction joints and exceptional care in placing will be required if leakage is to be avoided.

2. THE TWO COMMON PROCEDURES FOR TUNNEL LINING

Generally a tunnel is lined by one of two common procedures: (a) The continuous advancing-slope method, or (b) successive bulkheaded sections.

(a) *The continuous method* is a 24-hour operation, presumably without interruption except for breakdown or holidays. (See Fig. 1.) When the operation is stopped by breakdown, the resulting cold joint is of necessity the final slope of the concrete. The surface may be worked and shaped for most advantageous later continuation of the work by thorough compaction and by making a smooth even line against the forms. (See Fig. 2.) When the four inches of concrete nearest the form is placed toward the form on a 2:1 slope, a long rock pocket at the joint is more readily avoided.

When the continuous operation is purposely stopped, either the sloping joint may be used or the slope advanced to a vertical bulkhead until the arch is completely filled. Although the vertical bulkhead is

TION OF CONCRETE ON A SLOPE IN THE FORM

Result is that concrete must be run wet enough largely to compact itself with a minimum of working.

doubtless preferable theoretically, actually a properly prepared sloping joint is little different from many inadvertent cold or near cold joints occurring frequently on one or both sides of the tunnel due to delays from many causes. The sloping joint is also less expensive and work may be resumed with less difficulty in avoiding imperfections, particularly when the concrete is being placed by means of an air gun.

(b) *The bulkheaded section* is generally a length which can be filled safely each day in something less than one shift. Lengths of from 30 ft. to 120 ft. are not uncommon depending on size of tunnel, capacity of equipment, and whether the invert and arch are lined in one operation.

In many cases, even in bulkheaded sections, the advancing-slope method is used whereby the concrete is discharged at the top of the arch. After enough concrete is discharged at the top of the arch at the beginning of the section to establish the slope, concreting advances the same as in the continuous method until the arch is filled to the new bulkhead. When the air gun is used it has been found helpful in reducing rebound and segregation, when commencing to place concrete in the next section, if the last three feet of the top of the arch is left unfilled. (See Fig. 4A.) If a pocket is not left, one will be cut a foot or more deep in the green concrete by the terrific blast of the air gun, and in the meantime considerable separation will have occurred.

In other cases between bulkheads (Fig. 4) the side-wall concrete is placed directly by hand or other means in approximately horizontal layers to such an elevation as is practicable. The overhead arch portion is completed by means of either the air gun or the concrete pump. A horizontal, more or less cold joint between the wall and arch concrete is almost inevitable unless commencement of the arch placing immediately follows the last lift of the side-wall concrete, and unless the section is comparatively short.

3. THE CONTINUOUS ADVANCING-SLOPE PLACING METHOD—PRO AND CON

The continuous advancing-slope placing method (Fig. 1) has the following *advantages* and *disadvantages:*

(a) *Points in favor* of the continuous method are: (1) Widely reputed low first cost; (2) No necessity for moving rig and discharge pipe except in one forward direction; (3) Simplest and most routine in organization and operation of lining methods; (4) Most tunnel concrete men are familiar with it; (5) Not as many openings required in forms; (6) No bulkheads to install or remove.

(b) *Objections and disadvantages* of the continuous method are:

(1) Since the concrete advances on a 3:1 to 4½+ :1 slope, the concrete is not placed in horizontal layers and all efforts to work the con-

crete result in flattening the slope (Fig. 3). Consequently more forms are needed and the concrete becomes loosened and unconsolidated above the point of working.

(2) Efforts to work the concrete cannot be more than partially effective because concrete then moves laterally on the slope instead of vertically downward into a condition of greater consolidation. Consequently internal vibration cannot be used effectively and the result is the necessity to use a concrete of relatively wet consistency requiring a minimum of working in order to fill in the forms completely and get satisfactory appearance.

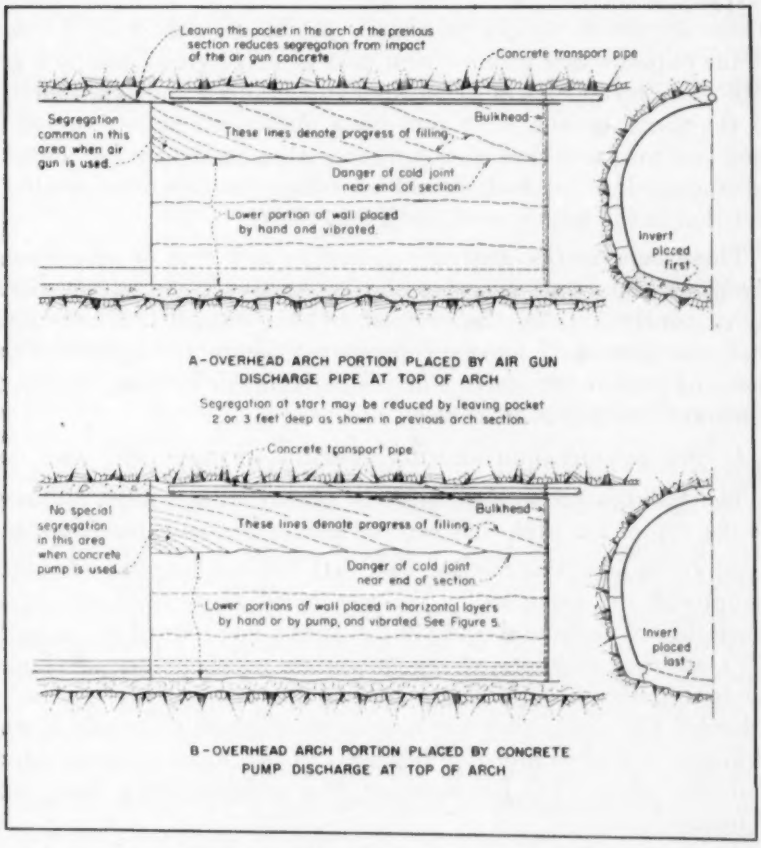
Leaving this pocket in the arch of the previous section reduces segregation from impact of the air gun concrete
Concrete transport pipe
Segregation common in this area when air gun is used
Bulkhead
These lines denote progress of filling
Danger of cold joint near end of section
Lower portion of wall placed by hand and vibrated
Invert placed first
A-OVERHEAD ARCH PORTION PLACED BY AIR GUN DISCHARGE PIPE AT TOP OF ARCH
Segregation at start may be reduced by leaving pocket 2 or 3 feet deep as shown in previous arch section
Concrete transport pipe
No special segregation in this area when concrete pump is used
Bulkhead
These lines denote progress of filling
Danger of cold joint near end of section
Lower portions of wall placed in horizontal layers by hand or by pump, and vibrated See Figure 5
Invert placed last
B-OVERHEAD ARCH PORTION PLACED BY CONCRETE PUMP DISCHARGE AT TOP OF ARCH

FIG. 4—TUNNEL LINING BY SUCCESSIVE BULKHEAD SECTIONS

See Fig. 1 for condition when entire section is placed from pipe discharge at top of arch by advancing slope method.

Note bulkhead attached to face of form so that form for next section may be set before all of bulkhead in arch is removed.

(3) The wetter concrete requires 10 to 20 per cent more cement for equal strength and is actually less desirable.

(4) The wetter concrete having a higher water content will have more frequent and wider cracks when drying occurs.

(5) With no provision for contraction joints, transverse cracking is inevitable at intervals from 15 to 100 ft., depending on degree of restraint.

(6) Where leakage is of consequence, cracks must be calked and somehow repaired at a considerable expense (in reality chargeable to the cost of this method).

(7) Slope marks, often long unsightly imperfections, are common. These are due to delays, principally in changing the flow of concrete to the opposite side before a near cold joint is formed, and to a generally wide-spread reluctance to work tunnel lining concrete thoroughly in the forms because of a prevailing impression among tunnel foremen and workmen that such concrete, when placed by the continuous advancing-slope method, should be sufficiently rich, over sanded and wet that it will largely work itself into place.

These slope marks, generally caused by new runs of concrete down the slope after some time has elapsed, can be made unobjectionable by diligently working the concrete but the hazard is always present that some unworked, harsher concrete will occupy these planes of weakness and require expensive and unsightly repair or cause a joint that is always susceptible to leakage.

4. THE BULKHEADED SECTION PLACING METHOD—PRO AND CON

The bulkheaded section, entirely placed from a pipe discharging at the top of the arch, has certain advantages and disadvantages.

(a) *In its favor* this method has: (1) The common use of standard equipment the same as in the advancing slope method. (2) No possibility of horizontal cold joint at spring line toward end of section. (3) Construction joints at the bulkheads which reduce or eliminate random transverse cracking according to their spacing. This is its principal advantage over the continuous method, especially if watertightness is desired and it is intended to put water stops in the construction joints. (4) Requirement of a comparatively short length of forms.

(b) *Against this method* are: (1) All the objections to the continuous method listed in 3(b) except as cracking is limited by and concentrated at the construction joints. (2) Areas very much higher in segregation and sloping imperfections at the start of the side walls

in *each section* when the air gun is used. (See Fig. 1.) (3) The costs of installing and removing bulkheads.

5. A MODIFICATION OF THE BULKHEADED SECTION PLACING METHOD

Where applicable a preferable bulkheaded section is placed in horizontal layers as high as possible in the side walls by any of several methods depending on size and quantity factors, and the arch is placed either by the concrete pump or air gun. (See Fig. 4.) This method makes possible much earlier stripping of the lower portion of the bulkhead; and for this reason primarily it has been suggested that, considering the superiority of this lining and control of cracking and nearly as good cost and progress records, use of this method could be justified in preference to continuous advancing-slope method. A 24-hour program as follows has been suggested as being practicably applicable to this procedure:

8 hours—Placing concrete in prepared section.
4 hours—Remove, clean, and place forms.
8 hours—Placing concrete in prepared section.
4 hours—Remove, clean, and place forms.

At least two sets of collapsible forms would be required which could be moved forward through the forms in place. The length of section and the mixing and placing equipment should be so planned and coordinated that under favorable conditions the section can be placed in six hours. This leaves a two-hour margin for trouble, delay, or an extra heavy or difficult section. Crews would have to be capable of either form preparation or concrete placing but many bulkheaded-section lining jobs have been successfully executed on this basis.

After the forms are stripped, moved, cleaned, and oiled an hour is ample for setting them in place. Assuming the preceding section required the full eight-hour placing period, a minimum interval of three hours is left for the last placed arch concrete to set sufficiently to remain in place when the upper portion of the bulkhead is removed. Under many conditions this interval will be sufficient. If there is occasion to leave the arch back 2 or 3 feet as shown in Fig. 4, or if the interval since placing is longer, due to less than 8 hours placing time, it will not matter if the concrete is not quite hard in a 3-hour minimum period. Other remedies for this situation would be to leave the previous arch back 2 or 3 feet regardless of the arch placing method, or, if the difficulty is expected regularly, for instance due to cold concrete, low heat cement, or both, or where water stops must be taken into consideration, the bulkhead can be designed to attach to the face

of the form so as not to interfere with attachment of the next form section, and the upper portion can then be removed later (but with somewhat more difficulty) working through portholes provided for this purpose at the back end of the arch forms. This could readily be done when it would not interfere with placing concrete in the side walls during the first several hours of the placing shift. The new bulkhead can be installed ahead of concrete on the concrete placing shift.

(a) *Advantages* of such a procedure are: (1) Separation of the concrete in placing the side walls may be entirely avoided. In the arch, separation is greatly limited by the much faster rate at which the arch is filled when the walls are not being placed at the same time. (2) No horizontal movement of concrete is required in the side walls and much less occurs in placing the arch than otherwise. (3) All efforts in working the concrete in the side walls result in consolidation without flow. (4) Internal vibrators may be used very effectively when necessary access is provided. (5) Thus 3 to 4-inch slump concrete instead of 6 to 7-in. (or wetter) may be equally well placed. (6) Consequently similar strengths may be obtained with 9 to 17 per cent less cement. (7) All work-plane joints in the walls may be eliminated or reliably and expeditiously taken care of. Even a cold joint at the last end of the horizontal joint between the side wall and arch may be largely avoided if operations are organized so that arch concreting follows promptly and is done without delay after the walls have been placed; otherwise a mortar batch should be used. (8) Fewer or no unsightly shrinkage cracks due to the effect of the bulkhead construction joints, depending on the length of the sections, and due to the lower water content possible in the concrete. (9) Where water stops are desired in contraction joints, a procedure is already established in which they can be included at small cost in addition to the material. (10) Where leakage through haphazard cracking, imperfections, and cold joints may be a serious matter, the procedure permits its control by proper spacing of water stops and by facilitating the placement of high quality concrete. (11) It is particularly well suited to the placing of impermeable, dense, highest-quality concrete such as would be necessary in cases where corrosive waters or gases must be resisted. (12) Ample time is provided between placing shifts for all servicing, and routine maintenance and repair of equipment, thereby insuring more uniform operation and a minimum of delays when concreting is actually under way. These common delays, largely avoidable by thorough preparation in the 4- to 6-hour interval before each concreting shift, are the cause of many of the imperfections

described in 3(b) as resulting from the continuous advancing-slope method. It is in the avoidance of these delays that better lining can be placed nearly as fast and as cheaply by this method as by the continuous advancing-slope method. (13) In general, this method avoids the objections to the continuous advancing-slope method enumerated in 3(b) and the objections to the use of the latter method in bulkheaded sections described in 4(b).

(b) *Disadvantages* are that the following modifications of common tunnel lining equipment are required: (1) More openings in the forms are required for working the concrete. (In forms for large tunnels, when more than one row of ports are required, the openings should be staggered. They should be circular and at least 20 in. in diameter.) (2) Placing equipment must be arranged to move readily back and forth several times when placing the side wall concrete. (3) Form alinement struts to ground cannot be used above the horizontal center line. (4) Placing equipment as it discharges into the walls must be arranged to drop the concrete practically vertically into the forms at any point. The equipment must be capable of being transfered quickly from one side to the other several times in placing each section. This may be accomplished by an arrangement to carry the concrete from the pipe normally discharging inside the forms at the top of the arch to the desired point in the walls, or by an arrangement to put the concrete by hand or by chutes (with proper end control) from cars or agitators through various openings in the forms.

In view of the need to have on hand special equipment for placing the concrete in the arch, it appears that a suitable adaptation of the arch placing equipment would be most efficient for placing concrete in the walls. Among the most suitable of such adaptations would be quickly attachable and detachable light flexible hose sections connected at the top of the arch to end of the discharge pipe from the concrete pump. Such hose pipe would be preferably of the same height as the diameter of the discharge pipe and 2 or 3 inches wider, lightly reinforced so as not to collapse or kink, readily transferred from side to side, and be either provided in several appropriate complete lengths to reach the different lifts in the side wall, or, in quickly detachable articulated sections. (See Fig. 5.) Such sections would be most expedient where space is ample for the connections. (Fig. 5A). Where it is not, interchangeable sets of full length hose could be used. (Fig. 5B). This equipment cannot be used in connection with pneumatic methods of placing. When the air gun is used there is the possibility of interference with patents or payment of a small royalty

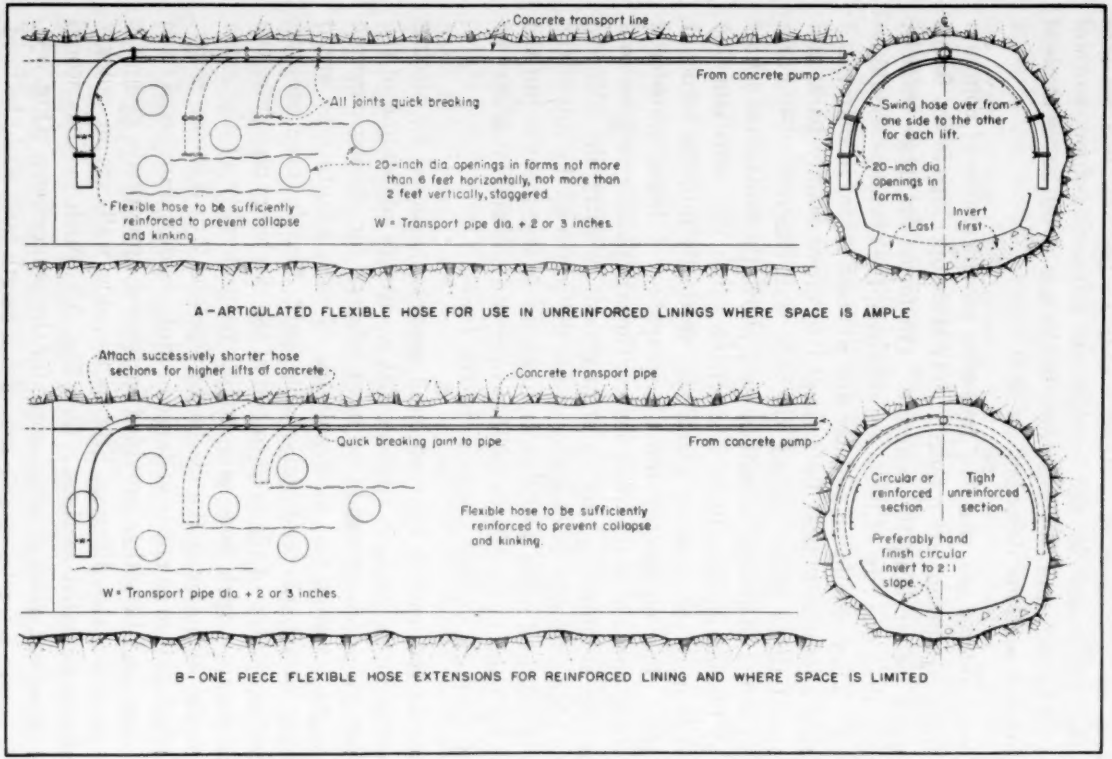
Concrete transport line
From concrete pump
All joints quick breaking
20-inch dia. openings in forms not more than 6 feet horizontally, not more than 2 feet vertically, staggered
Flexible hose to be sufficiently reinforced to prevent collapse and kinking
W = Transport pipe dia. + 2 or 3 inches
Swing hose over from one side to the other for each lift.
20-inch dia. openings in forms.
Last
Invert first
A - ARTICULATED FLEXIBLE HOSE FOR USE IN UNREINFORCED LININGS WHERE SPACE IS AMPLE
Attach successively shorter hose sections for higher lifts of concrete
Concrete transport pipe
Quick breaking joint to pipe
From concrete pump
Flexible hose to be sufficiently reinforced to prevent collapse and kinking.
W = Transport pipe dia. + 2 or 3 inches
Circular or reinforced section
Tight unreinforced section
Preferably hand finish circular invert to 2:1 slope
B - ONE PIECE FLEXIBLE HOSE EXTENSIONS FOR REINFORCED LINING AND WHERE SPACE IS LIMITED

FIG. 5—EQUIPMENT FOR CONVEYANCE OF CONCRETE INTO TUNNEL SIDEWALLS FROM CONCRETE PUMP DISCHARGE AT TOP OF ARCH

NOTE—This equipment cannot be used with the air guns.

for use of side chutes from a baffle-box. (5) This method will be difficult if not impossible to use in small tunnels with thin or reinforced linings, or both.

6. EQUIPMENT FOR PLACING CONCRETE IN TUNNEL LININGS

The air gun or the concrete pump. Which should be used is a question commonly asked when the matter of tunnel lining equipment and methods is under consideration. In the foregoing discussion of lining methods, reference has occasionally been made to one or the other of these pieces of placing equipment where it was especially related to the method described. In this section the merits and advantages, and the faults and objectionable features of each of these devices will be given.

The pneumatic gun, which is generally arranged to discharge through a pipe longitudinally into the crown of the tunnel arch, in most types consists of a horizontal steel tank or cylinder containing a system of compressed air jets, and in some cases also an air-operated plunger, all arranged to eject a charge of concrete which has been introduced by gravity through a hatch in the top of the gun.

(a) *Advantages and favorable application.* (1) Compressed air power is already on hand and available for other purposes upon completion of tunnel excavation. (2) Most foremen and operators are familiar with pneumatic placing equipment and procedure. (3) Good placing capacity when ample air storage and adequate mixing capacity are favorable. (4) Repeatedly demonstrated to place a fairly good lining at reasonable costs when cement content and concrete quality in comparison with other types of modern concrete work are not considered. (5) Can be used with minimum difficulty in connection with the commonly used continuous advancing-slope method of lining. (See Fig. 1.) (With this method the concrete pump, however, may be used to obtain equal if not better results.)

(b) *Disadvantages and unfavorable applications.* (1) The air gun separates concrete badly in starting each new section of lining separated by contraction or construction joints. (See Fig. 1 and 4A.) The larger the tunnel the greater is the trouble from this cause. According to an experienced inspector during lining of the arch portion of some large water tunnels, concrete placed at the start of each section by the air gun was subjected to "complete segregation of the component parts of the mixture due to the coarse aggregate ricocheting and coming to rest in various parts of the form. No amount of working could reassemble them in their proper combination." Separation of the

concrete by the air gun in lining the first sections of the Low Level Broadway Tunnel at Oakland, Calif., was so great that there were many rock pockets which leaked excessively and had to be cut out and repaired with gunite. This highly unacceptable performance of the air gun at the start of this lining, which is by no means an isolated case, resulted in an early switch to the concrete pump, which served successfully and satisfactorily for the remainder of the work. As completed, this tunnel is an excellent example of good results obtained in lining an important highway tunnel by means of bulkheaded sections with the concrete properly placed in horizontal layers up to the crown. (2) Because of the violence of its discharge, the air gun is not readily adaptable to placing concrete in horizontal layers in tunnel side walls. (See Fig. 5.) (3) Solid pressure packing of the concrete in the crown of the arch, sustained from sagging until setting holds it, is doubtful with the air gun, even though the pipe is well buried in the concrete, due to the necessary escape of large volumes of air. Solid packing is even more doubtful when the pipe is not buried for, though escape of air is not involved, there is nothing to sustain the concrete from sagging back between periods of discharge. (4) The intermittent discharge of the air gun is conducive to plugging and, particularly in filling the arch, or in general in connection with the continuous advancing-slope method, presents a condition unfavorable to the best results regardless of plugging. (5) The air gun causes a marked loss in slump between the mixer and the forms. Even when the side walls are hand placed there may be a 1 in. loss of slump, and with the concrete pump, a 1½-in. slump loss has been observed on a number of projects, due to the time effect in early stiffening of the concrete. But with the air gun this loss in slump has been found to be from 2½ to 3½ in. This means that enough more water must be used in air-gun concrete at the mixer to produce the slump necessary at the forms. Although some of this water may be removed by the air blast in placing, most of it is is still present in the stiffer concrete as indicated in the following groups of tests (Tables 1, 2 and 3) from several Colorado River aqueduct tunnels in all of which the aggregate was similar desert wash material. From these strength results it appears that this apparent drying of the consistency is not drying but stiffening due to whipping the concrete with air while shooting it into the forms. In all these tunnels concrete was placed by the continuous advancing-slope method and was of nominally the same essential consistency for workability and placeability in the forms. Note in all cases a much higher W/C ratio, even with one-half sack more

TABLE 1—5½ SACKS OR 1.375 BBL. OF CEMENT PER CUBIC YARD

Data	Concrete Pump				Air Gun
Tunnel	M Palms No. 2	Wide No. 1	Wide No. 2	7 Palms	Cottonwood
W/C ratio	.62	.61	.61	.62	.74
Str. p.s.i.	2760	2850	2870	3060	2270
No. tests	59	112	34	54	60

TABLE 2—6 SACKS OR 1.50 BBL. OF CEMENT PER CUBIC YARD

Data	Pump	Air Gun			
Tunnel	Yellow	W Eagle	Hayfield No. 1	Hayfield No. 2	Cottonwood
W/C ratio	.58	.67	.67	.67	.68
Str. p.s.i.	2910	2700	2610	2390	2500
No. tests	21	54	219	122	96

(Richer concrete used principally with air gun.)

TABLE 3—AVERAGES OF TABLES 1 AND 2

Data	Pump	Air Gun	Air Gun
Cement per cu. yd.	5½ sacks	6 sacks	6.6 sacks
Water per cu. yd.	5.03 cu. ft.	6.03 cu. ft.	5.94 cu. ft.
Avg. W/C ratio by wt.	.61	.67	.60
Avg. Str. p.s.i.	2885	2550	2950
No. test specimens	259	491	44

cement per yard (proving wetter concrete at the mixers), and corresponding lower strength, for the air-gun concrete regardless of any drying effect of air.

In Table 3 showing averages of Table 1 and 2 it is clear that although one-half sack more cement is used per cubic yard placed by the air gun, the strength is 335 p.s.i. less than that placed by the pump. That about another one-half sack extra (total 1 sack) was necessary for the air gun to place concrete of strength equivalent to the pump was indicated by the average strength of 44 specimens from the Cottonwood tunnel, containing 6.6 sacks per yard, which at 2950 p.s.i. is about the same as the 2885 p.s.i. strength of the 5½ sack pumped concrete.

It is evident that more cement for equal strength is required by the air gun.

(6) The air-gun concrete, for equivalent workability in the forms, may contain as much as *one cubic foot more water* per cubic yard of

concrete than that required in concrete placed under similar conditions by the concrete pump according to the combinations of W/C ratio and cement content reported for the above tests.

The *concrete pump* is a heavy duty, single acting, horizontal piston pump using a conventional crankshaft and connecting rod. The cylinder is horizontal with the outlet connection directly in line with the cylinder.*

(c) *In its favor* the concrete pump has: (1) A steady volume-building discharge which entirely avoids separation. (2) Lower power charge for operation than for compressors to supply air gun. (3) Ability to deliver unseparated concrete in horizontal layers because of low velocity of discharge. (4) A continuous flow of concrete which makes it less susceptible to plugging, thereby eliminating many cold joints and related imperfections, and the ability to keep the concrete in motion, which is favorable to best results and better filling, particularly in the arch and when used in connection with the continuous advancing-slope method. (5) Ability to pack the arch tightly when the pipe discharge is well buried because there are no large blast volumes of air which must escape. (Small intermittent shots of air, commonly used to clear around the end of the discharge pipe from the pump, interfere very little with packing the arch—in fact, materially assist in doing so, especially toward the shoulders. In some locations there is a trend toward the use of much more air with the pump than is necessary for best results. In addition to nullifying many of the advantages of the pump over the air gun, excessive use of air requires the added expense of maintaining an unnecessarily large system of air compressors and receivers.) (6) Ability to fill overbreak in the arch higher and tighter when the discharge pipe is buried 10 to 15 feet because of the sustained solid pressure behind the discharged material and because large volumes of air do not continually have to escape. (In one case in the Coachella tunnels, an inspector was able to get up into the cribbing in an extensive overbreak and actually see the concrete being elevated by the pump 13 feet above the discharge into the cribbed overbroken area. It is doubtful if an air gun could have accomplished this degree of filling. In open space above the forms, the impact of the air gun may thrust up the concrete a few feet, but it has no means of forcing the concrete higher and higher in the cribbing as has been done by the pump.) It is important that the discharge pipe from the pump be deeply buried if the arch is to be

*For further description see "Concrete by Pump and Pipe Line" by Chas. F. Ball. JOURNAL, Amer Concrete Inst., Jan.-Feb. 1936; *Proceedings* Vol. 32, p. 333. For an example of its set up in tunnel lining equipment see *Southwest Builder and Contractor*, November 1934.

tightly filled. This was clearly demonstrated on the Colorado River aqueduct, East Iron Mountain and Coxcomb tunnels. Both were lined by the same concrete pump and methods except that in the East Iron Mountain tunnel the pipe was not well buried, while in the Coxcomb tunnel it was. In the former the arch proved to be poorly filled but in the latter it was well filled. The only difference was the depth the discharge pipe was buried in the soft arch concrete. The poorly filled Iron Mountain arch is some times unfairly used to show the limits of the concrete pump when actually the method and not the equipment was at fault. (7) The concrete pump permits up to *one cubic foot* less water per cubic yard of concrete at the mixer for the same degree of workability in the forms. This results in either greater strength, or cement economy at the same strength. The lower water content also reduces the shrinkage on drying. (8) Last, but by no means least, the concrete pump has in its favor freedom from the following serious objections to the air gun: Separation at free discharge, high water content due to large loss of slump, higher cement requirement, and intermittent flow.

(d) These are *disadvantages* of the pump: (1) Comparative unfamiliarity of construction men with the machine, its operation, maintenance, and repair. (This is becoming less a fact with the increasing use of the pump.) (2) Slightly higher first costs for equipment. (3) One or two periods each 24 hours for thorough servicing and maintenance operations are required to prevent delays which, when they occur, are generally longer than with the air gun. (4) Equipment of this kind being made so far by but one manufacturer is difficult to specify without encountering complaints of favoritism and monopoly. However, since for several years the pumps have been sold outright, a number of used ones are in the market. These should serve to keep the price of new or used pumps within reasonable limits and prevent serious hold-up on account of a specification requirement. (5) Higher maintenance and repair costs particularly on account of the necessity for routine renewal of valve faces and cylinder sleeves. Since very considerable improvement has been made in the wearing quality of these parts in the last few years, judgment of their performance should not be based on that of several years ago. (6) The trickle of water required for lubrication of the pump cylinder may enter the concrete as the piston seal and cylinder sleeve become worn. (7) When a plugged pipe does occur it generally results in longer delay than a plugged line from an air gun because there is usually more concrete in the pipe. However, plugging can be largely prevented if

wet separated mixes, harsh lean mixes, poor mixing, and intermittent supply are avoided, and the pump is well serviced and maintained so that long stops with a full pipe line are unnecessary.

7. CRACKING AND LEAKAGE AS AFFECTED BY LINING METHODS

In addition to the foregoing considerations of method only, there are several points relative to cracking and leakage which should receive consideration in connection with this subject of tunnel lining methods. Briefly these are:

(a) In simple flow line tunnels where air-gun, continuously-placed concrete may be considered adequate and appear to be lower in cost, a shrinkage cracking and leakage problem usually results, the solution of which, if necessary, is seldom charged against the estimated savings of the continuous advancing-slope method of lining. If it were, the apparent economy of the continuous method would be considerably reduced and might disappear altogether.

(b) The continuous method, whether air gun or pump is used, will result in shrinkage cracking, but in some cases this will not matter.

(c) With the same mix at similar consistencies in the forms, air-gun placed concrete will have the highest unit water content and for this reason may be expected to crack more frequently and widely.

(d) Where cracking resulting from use of the continuous advancing-slope method is objectionable from the standpoint of leakage, it is believed cheaper and more satisfactory in the long run to control this cracking within self-closing and self-healing limits (see h) rather than permit ordinary cracking to occur and then attempt to repair it by grouting, calking, or other means.

(e) Cracking of continuously placed tunnel lining is due to the combined effect of shrinkage caused by drying, and that caused by the usual drop from a high setting temperature to normal.

(f) That portion of the cracks due to drying of the concrete will close when the concrete becomes saturated in use.

(g) That portion of the cracks due to temperature drop and cold flowing water may not be expected to close and presents a source of leakage and trouble.

(h) Thermal control of the concrete in the following particulars will largely correct this latter difficulty (g) and at the same time vastly improve working conditions in the tunnel: (1) Use of low-heat cement. (2) Use of efficient mixes and methods requiring no excess cement. (3) Use of mixing water at 35° F. (4) Use of cement that has been permitted to come down to normal temperatures. (5) Use of aggregate

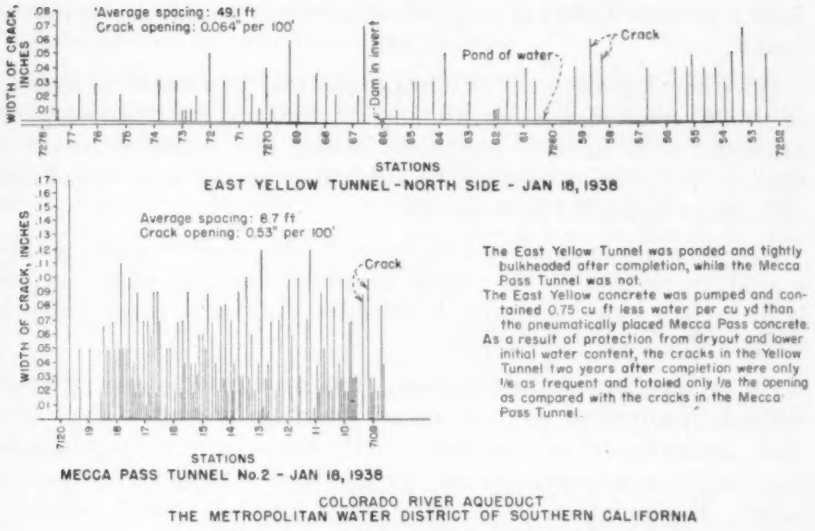
WIDTH OF CRACK, INCHES
Average spacing: 49.1 ft
Crack opening: 0.064" per 100'
Dam in invert
Pond of water
Crack
STATIONS
EAST YELLOW TUNNEL - NORTH SIDE - JAN 18, 1938
Average spacing: 8.7 ft
Crack opening: 0.53" per 100'
Crack
STATIONS
MECCA PASS TUNNEL No.2 - JAN 18, 1938
The East Yellow Tunnel was ponded and tightly bulkheaded after completion, while the Mecca Pass Tunnel was not.
The East Yellow concrete was pumped and contained 0.75 cu ft less water per cu yd than the pneumatically placed Mecca Pass concrete.
As a result of protection from dryout and lower initial water content, the cracks in the Yellow Tunnel two years after completion were only 1/8 as frequent and totaled only 1/8 the opening as compared with the cracks in the Mecca Pass Tunnel.
COLORADO RIVER AQUEDUCT
THE METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA

FIG. 6—CRACK SURVEYS, SHOWING BENEFICIAL EFFECT OF MAINTAINING PROTECTION FROM DRYOUT AND LOWER INITIAL WATER CONTENT IN REDUCTION OF NUMBER AND WIDTH OF CRACKS

that has been cooled at least to 70° F. and preferably to 50° F. or lower. (6) Use of water curing with generous (but not wasteful) application of water no warmer than the surface of the concrete and as much colder during the first 7 days as may be possible without producing surface checking.

(i) Shrinkage from drying in the interval between completion and water flow may be very greatly limited by protecting the lining from excessively drying conditions: (1) By forming a continuous pond of a few inches of water in the invert. (2) By bulkheading the portals tightly during the interval between completion of lining and operation as a water carrier. (3) By, during lining operations, creating ventilation by placing an outblower in a bulkhead in the portal where lining commenced so as to draw moist air from the curing section through the finished portion.

Results of such methods are indicated in Fig. 6.

(j) Although the procedures described in (h) and (i) may be expected to reduce leakage from continuous-method lined, flow-line water tunnels to within acceptable limits, it is doubtful, due to the vagaries of construction, if they would completely eliminate leakage, especially

from a pressure tunnel or from wet ground into a vehicular or railway tunnel.

(k) Where leakage out of or into a tunnel may be a matter of concern, it would seem mandatory to provide frequent contraction joints protected with suitable water-stop plates, and establish means of placing the concrete that will insure their proper embedment and subsequent successful performance.

(l) The gentle unseparated discharge of the concrete pump against a joint, in contrast to the violent separated discharge of an air gun, is by far the most conducive to effective watertight construction of water-stopped contraction joints.

(m) Where haphazard shrinkage cracks are undesirable, whether water is involved or not, and especially where protection from ultimate dryout cannot be provided, as in the vehicular or railway tunnels, such cracks may be avoided only by provision of frequent contraction joints. Depending on the restraint, these intervals should preferably be from 20 to 50 feet.

(n) The lower cement and water content concrete placed by the pump, which may be as much as one sack of cement and one cubic foot of water lower per cubic yard than in concrete of similar strength and workability placed by the air gun, is more favorable to the control of cracking because the shrinkage of such concrete is less.

8. LINING THE TUNNEL INVERT

Tunnel inverts are lined by a number of methods, depending on the size, shape, and purpose of the tunnel. The invert in large horseshoe-section water tunnels and in highway and railroad tunnels, when the invert is lined, is usually placed separately. In the smaller tunnels, it is common to find the entire section lined in one operation. However, the invert is sometimes placed first in circular tunnels.

Of all the devices used to place invert concrete as a separate operation, the one deserving the most whole-hearted condemnation is the use of some form of an air gun, shooting the concrete into a baffle and letting the concrete fall scattered and separated. Fortunately this is wholly unnecessary since slight modification of whatever equipment is used to charge the air gun (or other arch placing equipment) can be used to place the concrete directly in the tunnel invert. Care should be exercised that separation is avoided by dropping the concrete through two or three feet of vertical down-pipe at the end of the distributing chute or conveyor. In fact, aside from methods of shaping and finishing, the quality of the concrete in the invert largely

depends on the degree to which the well-known rules for proper handling and placing of concrete are carried out.

For shaping and screeding flat or long-radius inverts a standard paving and finishing machine is best where the job is large enough to afford it, because it will handle a more efficient mix containing minimum portions of water and mortar and largest practicable maximum size of aggregate. However, a slip-form is much preferable to forms in place for shaping short radius inverts, and is also preferable to the use of template screed strips and hand screeding. The mix required for successful use of the slip-form will be no less efficient than mixes which must be used with either forms in place or hand screeding. The slip-form followed by hand finishing produces a far superior quality of surface than the soft, pitted surface invariably obtained in a formed invert, and gives a truer shape than will ordinarily be obtained by hand methods.

A highly successful substitution of a slip-form for a formed invert was made in the monolithic construction of the 12-ft. 9-in. circular siphon sections in schedule No. 8 of the Colorado River aqueduct by Clyde Wood of Wood & Bevanda, contractors, in 1936. Although not in a tunnel lining, this use of a slip-form instead of a fixed-form panel is mentioned to indicate the possible practicability of using a slip-form for the invert of tunnels entirely lined in one operation, in which permanent quality and smoothness of the invert is an important consideration. Depending on the rate of progress and the rate of hardening, a slip-form of proper length could be readily adapted to jobs where the concrete was discharged at the top of the arch and the lining moved forward on a slope as in Fig. 1. Ordinarily a slip-form 15 to 20 feet in length would be adequate.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Cracks in Exterior Masonry Bearing Walls Occurring Where Concrete Roof and Floor Slabs Bear on the Walls*

By A. M. Korsmo and B. M. Thorud†

SYNOPSIS

This paper has to do with cracks which have developed in new buildings in which typical construction is of solid reinforced concrete floor and roof slabs supported on exterior masonry walls and interior beams and columns. In general, the cracks occur in horizontal joints of the external wall surfaces at corners of the buildings under concrete floor and roof slabs. The description of the phenomena is offered as a step toward solution of a general problem.

In a number of the housing projects of the Housing Division of the Public Works Administration and later transferred to its successor, the United States Housing Authority, a peculiar condition of cracking developed in masonry bearing walls. While these cracks were not a hazard to safety or especially detrimental to appearance or utility, they were regarded as a matter of concern, requiring repair after completion of the buildings. Because the causes of the cracks have not been conclusively ascertained, they have become a matter of much theorizing and investigation. It is the purpose of this paper to set forth in detail the nature of the cracks as observed on projects built under the P. W. A. Housing Division.

Occurrence of similar cracks has been observed and reported on many buildings other than Government-aided housing projects, where similar construction has been employed. Therefore, solution of the causes of the cracks and design of preventitive means of construction

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†Principal Structural Engineer and Senior Structural Engineer, respectively, U. S. Housing Authority, Federal Works Agency, Washington, D. C.

are of interest to the entire building industry. Much varied analysis and theorizing have been expressed and several papers [2] have been published concerning the formation of these cracks and proposing details of construction intended to prevent cracking.

The buildings were all of a typical construction consisting of solid reinforced concrete floor and roof slabs supported on exterior masonry walls and interior beams and columns. The typical building plan consisted of units 27 feet deep placed in one long row or arranged as Ts or Ls, or combinations of these. The slabs were usually 5 to 5½ in. thick for roofs and 5½ to 6½ in. for floors, reinforced as one way slabs in the 2 spans across the 27-ft. width; the interior support being one row of beams and columns, the end beam resting on the end wall. In general, expansion joints were omitted in these buildings which were up to 200 ft. long. In most of the buildings the slab spans were unequal, usually about 13 ft.-6 in. clear on one side of the interior beam support and 10 ft.-6 in. clear on the other side. The slabs all had simple 7½ in. bearing on the walls. Slab reinforcing generally consisted of ½ or ⅝ in. bars, with alternate bars bent up over supports.

The exterior walls were 12 in. consisting of 4 in. brick facing and 8 in. backup of either clay tile or concrete block. Most projects had clay tile backup, but cracking has occurred with equal frequency for both clay tile and concrete block backup. The walls were specified to be laid with back of face brick parged, face shells only of backup bedded in mortar and face brick fully bedded; vertical joints of the facing completely filled with mortar, bricks buttered and shoved into place; mortar 1:1:6 portland cement lime and sand or 1:3 masonry cement and sand, generally of 800 p.s.i. strength in 28 days.

The cracks in last instance occur in a horizontal joint of exterior wall surface at corners of the buildings, approximately at the underside of the concrete floor and roof slabs, shown at "A" in Fig. 1. If a membrane flashing has been used as indicated at "C," the crack usually occurs in the face of the horizontal joint in which the flashing is embedded as at "B."

The cracks occur most frequently at roof slabs and progressively less frequently at succeeding floor levels below the roof. Usually, the face of the brick above the cracks projects beyond the face of the brick below, as at "G," Fig. 1. The cracks extend varying lengths from the corner of the wall, generally not less than 4 ft. and occasionally as much as 20 ft. or more. The average length is probably about

(2) "Avoiding Cracks in Building Walls," by Fred. N. Severud, *Eng. News-Record*, July 6, 1939. "Low Cost in Low Cost Housing," by Elwyn E. Seelye, *Eng. News-Record*, July 20, 1939.

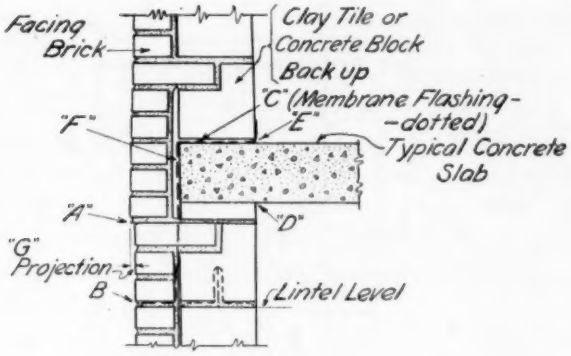

FIG. 1

6 ft. Slight openings in the vertical joints often appear above the crack indicating tension in the exterior face. The projection of the brick above the crack varies from maximum at the corner of the wall to no projection at the termination of the crack. It is not definitely known whether the deflection of the wall above and below the crack occurs as shown in Fig. 2 or as in Fig. 3. Occasionally, the crack extends a short distance in one joint and then continues in the joint above or below, in which case there is often a crack in the vertical joint near the offset in horizontal joint cracked. In some few instances the crack is stepped diagonally over several horizontal joints as indicated in Fig. 4.

The projections of the brick above cracks at the corner of the building vary from very slight to probably rare maximum projections of ¾ in. They average approximately ¼ in.

The projections appear to bear no relation to the length of the walls, i. e., projections in the direction of the short wall at the ends of the buildings appear to be as great as in the direction of the long walls.

There appears to be no separation between slab and inner faces of the wall as at "D" or "E," in Fig. 1, especially at "D" where separation would be readily apparent. This seems to indicate that the wall below the slab usually moves in or out with movement in the slab, i. e., that the bond of the slab to the wall at the bearing is sufficient to cause the slab and the wall to move together. In a few instances there was evidence of a slight inward movement of the slab with relation to the inner wall surfaces at the corner, or vice versa, the wall moving out with respect to the slab.

In several instances where the wall has been opened up at cracks, there was found to be a definite space between the edge of the slab

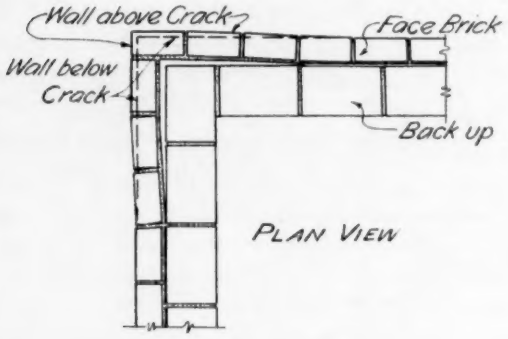

FIG. 2

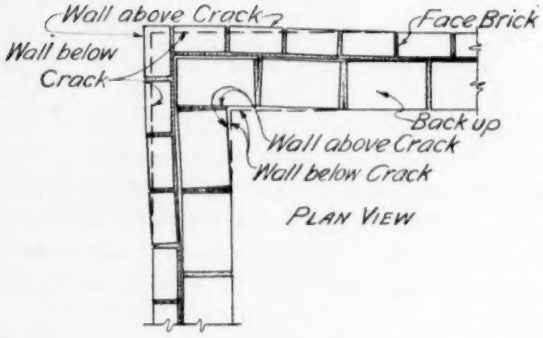

FIG. 3

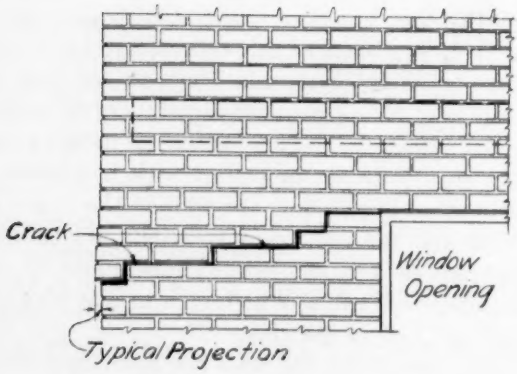

FIG. 4

and the brick facing, as at "F" in Fig. 1, indicating that the brick facing has moved out away from the slab or that the slab has moved in away from the brick facing. It may be that both of these movements occur simultaneously.

The cracks occur most frequently at roof slabs and progressively less frequently at succeeding floors below the roof. All corners do not develop cracks and at many large projects, no cracks have become apparent in two to three years since completion of the construction.

A typical distribution of cracks at a project where excessive cracking occurred was as follows:

4-Story Apartment Buildings, irregular shape:

Total number of bldg. corners at roof	185
" " " " " each floor	108
Corners cracked at roofs	164
" " " 4th floor	55
" " " 3rd "	29
" " " 2nd "	8

3-Story Apartment Buildings, irregular shape:

Total Number of bldg. corners	74
Corners cracked at roofs	66
" " " 3rd floor	45
" " " 2nd floor	14

2-Story Row House Buildings, rectangular shape with short offsets in long walls:

Total number of bldg. corners	64
Corners cracked at roofs	60
" " " 2nd floor	48

In some few instances cracks appeared at the lower floors and not at upper floors. Some cracks at roof continued for full extent of walls, some of which were 178 ft. long, but this was very unusual and not typical of most projects. They never seemed to exceed 30 ft. in length at a floor and this, too, was not typical of most projects, where they rarely exceeded 10 ft. long.

As to the number of projects reporting cracks: of 43 projects having similar construction, 14 have reported cracks at roof and floor levels, and 9 more have reported cracks at roof levels only. Of these, only 7 had large frequency of cracks.

However, large frequency was experienced in various sections of the country, and all cracks were of a similar type, which seems to indicate that a definite relative movement of some nature causes a strain which often breaks the bond of the mortar course. The exact nature of the movement has not been definitely ascertained. It appears that the extent of the cracking has often been too great to be

attributed only to expansion or contraction of the concrete slabs or longitudinal movement in the walls. The cracks most often seem to occur from 2 months to a year after construction has been completed.

The scope of this paper does not include any discussion of the various theories of causes of the cracks but the following observations may be of value in indicating these causes:

1. Careful check of foundations has indicated no evidences of movements which could contribute to forces causing cracks.

2. The slabs developed typical cracks diagonally across the corners, approximately 3 to 8 feet away from the inside corners of the buildings. These cracks appeared to be due to tensile forces, at least in part, as some of the cracks are sufficiently open, to allow asphalt roofing to flow through the slab. Very often these cracks develop where no cracks appear in the walls, and at many of the corners where walls are cracked, the slabs were not cracked.

3. In almost every instance where cracks occur at the corners, either in the walls or the slabs, and the length of the building, or wing of the building, exceeded 75 feet, one or two cracks through the slabs occurred at right angles to the long direction of the building or wing. These were located at the approximate center or third points of the lengths of the building or wing and generally extended across the full width of the building. They often occurred at or near openings in the floor slabs.

To verify or disprove the various theories which have been propounded to date, careful and frequent measurements of relative movements in the slabs and walls are now being taken on several projects recently completed.

Discussion, to close in February 1941 JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1940.*

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Discussion of a paper by A. M. Korsmo and B. M. Thorud:

Cracks in Exterior Masonry Bearing Walls Occurring Where Concrete Roof and Floor Slabs Bear on the Walls*

BY DOUGLAS E. PARSONS AND OSKAR SCHREIER

BY MR. PARSONS†

The authors deserve the thanks of the building profession for this detailed description of a type of defect which has been observed in many buildings, both commercial and Governmental. The paper is of exceptional value, because the observations reported are based upon a large number of careful examinations of buildings, many of which are of substantially the same shape and design. The authors, wisely I believe, refrain from discussing at length the several theories of causes of the cracks in the masonry walls and have confined their report to a presentation of pertinent facts.

Inasmuch as similar cracks have appeared in commercial structures, it is clear that they are not the direct result of construction practices peculiar to the Housing Division of the Public Works Administration. Neither climate, kinds of portland cement, aggregate, masonry mortar, or masonry unit can alone be responsible, because there are examples of structures which cracked and did not crack for each of the several types of these. Although it seems likely that weather conditions during construction and soon thereafter, as well as the speed and sequence of erection of the various portions of the structure, would influence the tendency for the formation of the cracks, data seem to be insufficient to establish a clear correlation with these factors. Cracks in the corners of walls occurred in some instances irrespective

*JOURNAL, Amer. Concrete Inst. Sept. 1940; *Proceedings* V. 37, p. 49.
†National Bureau of Standards, Washington, D. C.

of whether or not there was spandrel flashing. One of the outstanding facts indicative of the cause of the cracks was the tendency for them to be most frequent and widest at the roof and to become less frequent and narrower for the lower floors.

At the suggestion of the authors, and with the financial assistance of the U. S. Housing Authority, Federal Housing Administration, and the Home Owners' Loan Corporation, the National Bureau of Standards erected a small structure designed by Messrs. Korsmo and Thorud to be similar to a portion of one of the typical Housing Division buildings. The building was erected in September, 1939, and since that time measurements have been made at frequent intervals to determine the nature and magnitude of the deformations and displacements which caused the formation of corner cracks in such structures. The effects of shrinkage of the concrete and of changes in temperature have been similar to what would be expected from a knowledge of the properties of these materials, and horizontal cracks at the corners of the building developed as expected. Despite the fact that the movements responsible for the cracks probably have almost ceased by this time, the measurements are being continued in order to obtain additional data on the relations between these movements and changes in temperature. After these relations have been found, it may be possible to determine the relative magnitudes of the movements caused by changes in temperature and those produced by other causes. Although the information already obtained would be sufficient for a partial analysis of the causes of cracks and might be helpful in indicating methods of prevention, the data are too voluminous and yet not sufficiently complete to warrant a report as a part of this discussion. We expect to publish a report on the study as soon as the measurements have been completed.

BY MR. SCHREIER*

Based on thirty years experience as engineer and contractor in countries of widely varying climatic conditions, I should like to communicate my observations and ideas. Cracks rarely have their source in one case only: usually there are several factors involved. The important ones are: (1) unequal settlement of the foundation; (2) different behavior of the bearing construction under the load; (3) shrinkage; (4) temperature, especially frost. Allow me to consider the described constructions with regard to these four points.

(1) An equal settlement of the foundation is very important in buildings consisting of exterior masonry bearing walls and interior reinforced concrete columns. The foundations of the walls are usually continuous, in contrast to the individual footings of the columns.

*Engineer, New York, N. Y.

Although the calculated soil pressures may be equal, very many settlement graphs, drawn according to Professor Terzaghi's observation method, have shown differences in the settlements, especially when using two types of foundation. Corners are exposed to this influence mostly. Unequal settlements are one of the principal causes of cracks. In the cases described by the authors, this cause has been eliminated.

(2) The internal behavior of masonry bearing walls and reinforced concrete columns is unavoidably different, due to the greater compressibility of the mortar compared to the concrete. The settlements sum up and become greater as the height of the building increases. Therefore the cracks always appear first at the roof and at the upper floors. In my experience this combination of exterior masonry bearing walls and interior concrete slabs and columns should be avoided. This question was thoroughly discussed by the Building Code Committee in Vienna (of which I was a member) and the deliberations led to the conclusion that this construction should be avoided as far as possible. To attain equal internal behavior it is preferable to use reinforced concrete frames with spandrel walls. Very fine examples, executed in excellent manner in this country, prove that cracks can be avoided completely.

The described bearing wall of facing brick and clay tile or concrete block backup has also several weak points. The bond between face and backup by a header every sixth course only is insufficient. The floor loads are carried by the backup blocks, whose face shells only are bedded in mortar, another potential source of cracks. Membranes for insulation increase this tendency. In addition, the water absorption capacity of the face and backup materials, particularly of brick and concrete is different and this favors the formation of cracks. All these circumstances point to use this sort of masonry walls for low houses and spandrel walls only.

(3) The influence of shrinkage is so well known that it shall be mentioned only. But as far as I have observed, the use of expansion joints in house building constructions is not so general as should be expected according to the present theoretical knowledge. On the basis of my experience in designing and building some of the largest groups of low rent houses executed by the municipal Building Department of Vienna and Graz, all buildings longer than 100 ft. should be divided and in no case should the distance between the expansion joints exceed 80 ft. The joints must go throughout the building, the slabs as well as the walls.

It is very important therefore that the architects recognize the necessity of these joints and make proper provisions for them. Skillfully

disposed the joints will neither disturb the appearance nor the arrangement of the apartments. For engineers joints present no difficulties; the construction (a little more expensive perhaps) will be better, because too long continuity is always an additional danger.

(4) If cracking is reduced to the causes mentioned above, the described displacement of face at the cracks is typical for the influence of frost. Moisture penetrates through the cracks and the freezing pressure of the water causes such displacements. As corners are most endangered by moisture and frost, these typical movements are always observed at corners and at roofs. The projections shown at the inside of the corners can also be explained in this manner. I have observed roof parapets which were displaced precisely in the same way so much that they had to be pulled down and rebuilt.

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The Bond Strength of Rusted Deformed Bars*

By Bruce Johnston†
MEMBER AMERICAN CONCRETE INSTITUTE
and Kenneth C. Cox‡

SYNOPSIS

Results of about 420 bond pullout tests on deformed bar specimens of 78 different sizes or degrees of rust are reported herein. Three different series of tests were made in all. In the first series, bars were selected from a stock pile and were classified as to degree of rust. Although these bars were of the same nominal size and type it was found that small differences in the size of lug produced a greater difference in test results than the degree of rust. In the second series, deformed bars of different sizes cut from identical stock were stored both in a moist room and out-of-doors in an exposed position. The time of exposure was a variable and the maximum time for the out-of-door exposed specimens was fifteen months. The third series of tests consisted of bars exposed out-of-doors as a check test on the results of the second series.

INTRODUCTION

The effect of rust on the bond strength of rusted reinforcing bars is an important practical problem. The "Proposed Building Regulations for Reinforced Concrete" of the American Concrete Institute state in Sec. 504 "Metal reinforcement, at the time the concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond." This statement is open to the interpretation that if rust scale is of a type which will not destroy or reduce the bond the reinforcing bars may be considered acceptable.

*Received by the Institute April 5, 1940.
†Assistant Director, in Charge of Research, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania.
‡Formerly Concrete Reinforcing Steel Institute Research Fellow at Lehigh University. Now with Dravo Corporation, Pittsburgh, Pennsylvania.

FIG. 1—RUSTED BARS USED IN PART I LEFT TO RIGHT: A, B, C, D

In 1909 M. O. Withey reported in Bulletin 321 of the University of Wisconsin that a firm hard coating of rust improved the bond strength of plain round reinforcing bars as compared with as-rolled bars free from rust. Similar findings were reported by D. A. Abrams in Bulletin 71 of the University of Illinois Experiment Station, and more recently J. E. Shank* of Ohio State University has reported that red weather-rusted bars and ground rusted bars both gave considerably higher bond values than clean uncoated bars. A complete bibliography on the subject of bond in general is presented in the paper by H. J. Gilkey, S. J. Chamberlin, and H. W. Beal, "The Bond between Concrete and Steel."† The recent paper by C. A. Menzel "Some Factors Influencing Results of Pullout Bond Tests"‡ should also be mentioned in introducing the subject of bond tests.

TEST PROGRAM AND RESULTS—PART I

In this part of the investigation four different degrees of surface rust were tested for effect on bond value. As noted in the synopsis, the results of the tests in this part of the investigation were influenced more by variation in the size of lugs than by variations in rust coat. These results are included, however, as a matter of record. All the bars were $\frac{5}{8}$-in. round, intermediate grade, with transverse lugs. The bars were taken from a reinforcing bar fabricator's stock pile, care being taken to select bars for each of the groups tested having equal degrees of rust.

Judging the character of the rusted surface will usually be a field problem where elaborate equipment is seldom available. Attempts

*"Effect of Bar Surface Conditions in Reinforced Concrete", Ohio State Engineering Experiment Station News, June 1934, p. 9.
†JOURNAL, Amer. Concrete Inst., Sept. 1938; *Proceedings* Vol. 35, p. 1.
‡JOURNAL Amer. Concrete Inst., June, 1939; *Proceedings* Vol. 35, p. 517.

to develop a laboratory test to evaluate the degree of rust proved somewhat unsatisfactory. Photomicrographs of the rusted surface yielded little information. In another approach to the problem, ten-inch specimens were cut and weighed to the nearest tenth of a gram. The rust was removed with a buffer and the specimens again weighed. The differences were not very consistent and were too small and irregular to be of practical value. As a result of these preliminary experiments it was decided that the best method of judging the degree and character of the rust was by means of a visual examination, comparable to what might be made easily by a field inspector.

Fig. 1 shows the condition of the bars tested in this first group of tests. Descriptions of the bars are as follows:

Group A—The bars in this group had most of the original mill scale surface except for occasional, small yellow-brown rust spots. The bars were fairly smooth to the touch and the mill scale did not flake off easily.

Group B—The bars in this group had lost the mill scale color and were covered with a layer of firm rust. The color of the rust was dark brown and was slightly rough in texture. Rubbing with the hand did not remove the rust.

Group C—The bars in this group had lost all of the original mill scale surface and were pitted and covered by a firm hard rust of dark brown color. These bars were rough to the touch and occasionally granular pieces of rust could be rubbed off in pieces approximately $\frac{1}{64}$-in. in diameter. The pits covered the entire surface but were not very deep.

Group D—The bars of this group appeared to be in better condition than the bars of Groups B or C. The rust was yellow and gave the hands a yellow-brown stain. There were many places where the mill scale had remained intact. The rust was loose, soft, and crushed readily under small pressure.

The bond pullout test specimens made in this series of tests were made with the reinforcing bars in a vertical position and with the concrete settling against the direction of pull. The specimens were six inches in diameter and the bars were embedded in six inches of concrete.

Slip at the free end was measured by means of an Ames dial reading to 1/10,000 in. Fig. 2 shows a typical setup for a pullout test. A 50,000-lb. Riehle Testing Machine was used in making the tests, and the reinforcing bar was pulled at a rate of 0.05 in. per minute.

One group of twenty specimens was made with the bars as received, and in the second group the sixteen specimens were given a thorough

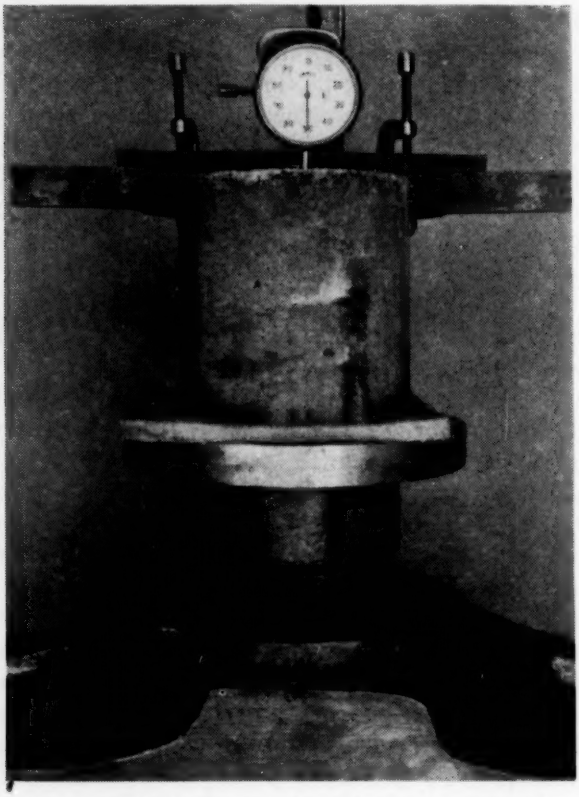


FIG. 2—TEST SET-UP FOR MEASURING END SLIP

brushing by hand. The concrete was designed to give a 28-day compressive strength of 4000 p.s.i. and the average obtained on eighteen 3 by 6-in. control specimens was 3930 p.s.i. The proportioning of the concrete was by trial mixes. The mix selected was as follows: cement 20.8 lb.; water 15.4 lb. (including absorption allowance); sand 58.5 lb.; $\frac{3}{8}$-in. coarse aggregate 35.5 lb.; and $\frac{3}{4}$-in. coarse aggregate weighing 53.0 lb. Twenty-four hours after molding, the specimens were placed in the moist room where they were cured for 27 days at 70° F.

Tensile tests were made for each group of bars with Huggenberger extensometer readings taken on two specimens and "drop of beam" yield point observed on the three other specimens. The results of these tests are shown in Table 1.

TABLE 1—TENSILE PROPERTIES OF STEEL USED IN SERIES I

Group	Unit Stress at Yield Point p.s.i.	Unit Stress at Ultimate p.s.i.	Per Cent Elongation in 8 in.
A	46,760	76,220	21.8
B	47,480	75,680	22.7
C	43,740	74,240	22.0
D	50,680	78,640	24.0

The test results of Part I of the investigation are tabulated in Table 2. The total pull on the bar for end slips of 0.00005, 0.0001, 0.0005, and 0.001-in. is recorded in the table for both the brushed and unbrushed specimens. Each figure in the table represents an average of five tests in the case of the unbrushed specimens and four tests in the case of the brushed specimens. Reference should be made to pages 4 and 5 for a description of the comparative rusted condition of the groups A, B, C, D.

TABLE 2—TEST RESULTS OF PULLOUTS IN PART I

Rust Classification	End Slip at Free End In Inches	Total Load (Average) "Unbrushed"	Total Load (Average) "Brushed"	Ratio: Brushed / Unbrushed
A	0.00005	5,066	5,130	1.01
	.0001	6,414	6,106	0.95
	.0005	9,370	8,876	0.95
	.001	10,362	9,870	0.95
	Unbr. 0.0285* Br. 0.0232*	13,368	14,127	1.06
B	0.00005	4,730	5,445	1.15
	.0001	6,086	6,670	1.10
	.0005	9,386	10,460	1.11
	.001	10,386	11,933	1.15
	Unbr. 0.0155* Br. 0.0232*	13,266	15,055	1.14
C	0.00005	5,626	5,510	0.98
	.0001	7,686	7,265	0.95
	.0005	12,654	12,652	1.00
	.001	13,572	13,935	1.03
	Unbr. 0.0431* Br. 0.0524*	16,672	20,052	1.20
D	0.00005	4,040	4,833	1.19
	.0001	5,206	5,708	1.10
	.0005	7,627	7,878	1.03
	.0001	7,983	8,555	1.07
	Unbr. 0.0625* Br. 0.0515*	12,706	13,578	1.07

*Average slip at ultimate.

Not much consideration should be given to the differences between the bond strength of the various groups, A, B, C, and D. The differences are influenced not only by the rust differences but also by small variations in the height of the transverse lugs, which may be noted in Fig. 1. Such differences in the size of lugs may be developed by reason of differential wear in rolls, in the same nominal size and type of bar.

The tests in Part I afford a comparison between the bond strength of brushed and unbrushed specimens, as affected by different degrees and classifications of rust. Brushing had no well defined effect upon the bond strength of the bars in classifications A and C. The bars in Group C were deeply pitted by rusting. The bond strengths of the bars in Groups B and D were raised an average of between ten and fifteen per cent by brushing, for low values of slip. The bars in Groups B and D were not as heavily rusted as those in Group C.

The results of these tests are inconclusive. The bond strength of bars with certain types of rust seemed to be benefited slightly by brushing, but there is no reason to believe that the unrusted bars of the same type would have higher bond strength than the same bars in the rusted but unbrushed condition.

TEST PROGRAM AND RESULTS—PART II

The results of Part I were inconclusive with respect to the net effect of varying degrees of rust upon the bond strength of rusted deformed bars.

In Part II of the investigation it was decided to expose bars for varying periods and thus provide different degrees of rust on identical specimens.

Six 20-ft. bars of each of the following sizes; ⅜-in.ϕ deformed, ½-in.ϕ deformed, ¾-in.ϕ deformed, 1-in. square deformed, and 1¼-in. square deformed were cut into two-foot lengths. The bars had transverse lugs and were of intermediate grade steel. The tensile properties of these bars are given in Table 3, which also gives the properties of similar bars tested in Part III.

TABLE 3—TENSILE PROPERTIES OF STEEL USED IN PARTS II AND III

Size of Bar	Unit Stress, p.s.i.		Per Cent Elongation		Per Cent Reduction in Area
	Yield Point	Ultimate	in 2 in.	in 8 in.	
Part II					
⅜ in.ϕ	50,000	75,000	30.5	22.6	55.6
½ in.ϕ	52,300	83,280	31.0	20.0	48.0
¾ in.ϕ	44,200	75,600	37.0	24.3	50.0
1 in.sq	41,900	67,510	40.5	27.4	38.5
1¼ in.sq	40,000	72,100	42.0	28.8	38.8
Part III					
⅜ in.ϕ	49,900	69,400	36.5	25.6	65.4
¾ in.ϕ	47,800	77,100	34.0	22.1	49.1
1 in.sq	47,600	74,560	39.0	26.0	42.2

Note—Each figure is an average of three tests

One specimen from each bar was used as a specimen to represent the unrusted state. One hundred and fifty specimens, or five specimens

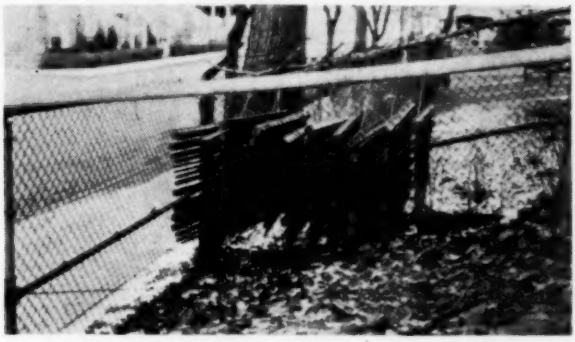


FIG. 3—BARS EXPOSED OUT-OF-DOORS

FIG. 4—BARS EXPOSED IN MOIST ROOM

from each bar, were wired into racks and placed on the north side of the Fritz Engineering Laboratory as shown in Fig. 3. The remaining bars were placed in the moist room at 70°F as shown in Fig. 4. The exposure started on December 15, 1937.

After periods of exposure of 3, 6, 9, 12, and 15 months the bond pullout test specimens were made. The concrete was designed to have an average compressive strength of 2500 p.s.i. at 28 days, with proportions as follows: cement 29.6 lb., water 27.8 lb., sand 92 lb., $\frac{3}{8}$-in. aggregate 61 lb., $\frac{3}{4}$-in. aggregate 122 lb. Sand and aggregate were weighed dry and weight of water included allowance for absorption. The average concrete strengths were 2580, 2610, 2370, 2402, 2720, and 2908 p.s.i. for 0, 3, 6, 9, 12, and 15 months exposure tests respectively. The depth of embedment was established at eight times the diameter of round specimens and eight times the minimum

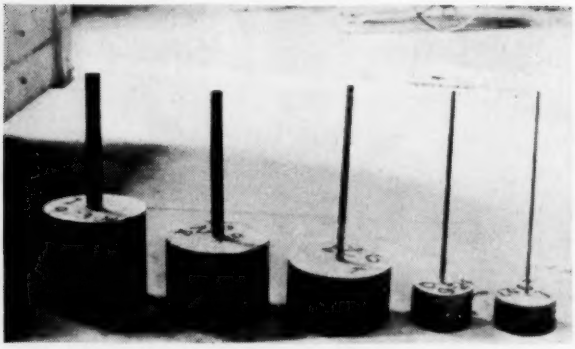


FIG. 5—GROUP OF PULL-OUT SPECIMENS FOR PART II

thickness of square specimens. Pilot tests indicated that with this depth of embedment the bars would not reach their yield point before being pulled from the concrete block. The concrete was placed so as to settle against the pull direction of the reinforcing bars, which were all held in a vertical position while the concrete was placed. Two diameters of specimens were used, namely, 6 and 10 in. The 6-in. diameter was used for the $\frac{3}{8}$-in.ϕ and $\frac{1}{2}$-in.ϕ bars while the 10 in. diameter was used for the $\frac{3}{4}$-in.ϕ, 1-in. square, and $1\frac{1}{4}$-in. square bars. These sizes were used to elininate splitting of the specimen before sufficient slip had taken place. Fig. 5 shows a typical group of pullout specimens made for this part of the program.

Six bond pullout tests in duplicate were made for each size and degree of exposure. Six 3 by 6-in. test cylinders were made to check the strength of each concrete mix. The concrete was rodded into place until sufficiently compact and free of voids. At the age of one day the pullout specimens and the cylinders were placed in the moist room, to be tested at the age of 28 days.

The rusted surface condition of the bars in this series of tests varied widely during the progress of the work. The zero month exposed bars were practically free of rust and were embedded in this condition.

The three months group had rust similar to that described for Group D, Part I. The bars exposed outside had a great deal more rust than those placed in the moist room. The small sized bars were quite thoroughly rusted over their entire area while the large bars had considerable areas relatively free of rust. The rust of the bars in the moist room was a little more yellow in color and somewhat more crumbly than those exposed outside.

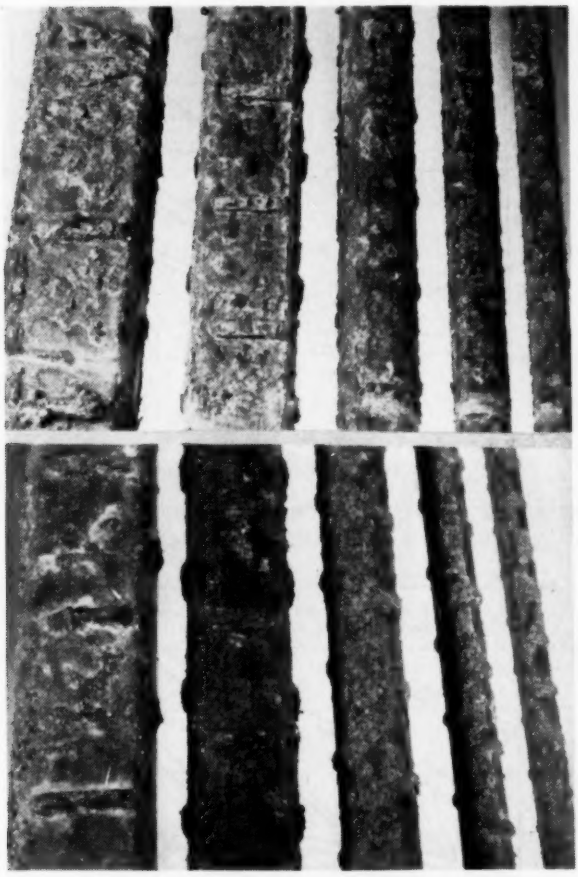


FIG. 6—BARS AFTER NINE MONTHS EXPOSURE. ABOVE—OUTDOOR EXPOSURE; BELOW—EXPOSURE IN MOIST ROOM

The six months group showed more rust than the three months group. The large sizes were now generally rusted and had reached a condition almost as bad as the small sizes had at three months. Again the outside rusted bars showed more rust than the moist room exposed bars.

The nine months rusted bars, shown in Fig. 6, had a very heavy coat of rust; the rust had become loose and flaky, especially on the bars exposed out-of-doors. Rust would crumble off under very small pressures and in general the bars at this stage had a very poor appearance. By the end of the twelve and fifteen months periods the bars had become progressively more rusted and were in far worse condition

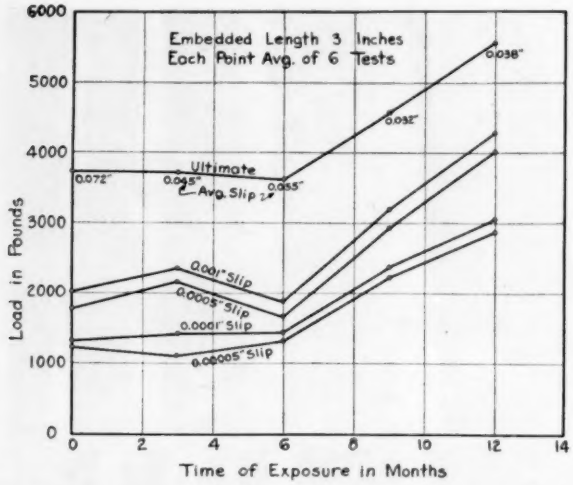


FIG. 7—EFFECT OF PRIOR EXPOSURE IN MOIST ROOM ON 28 DAY BOND STRENGTH OF $\frac{3}{8}$-IN. ϕ DEFORMED BARS

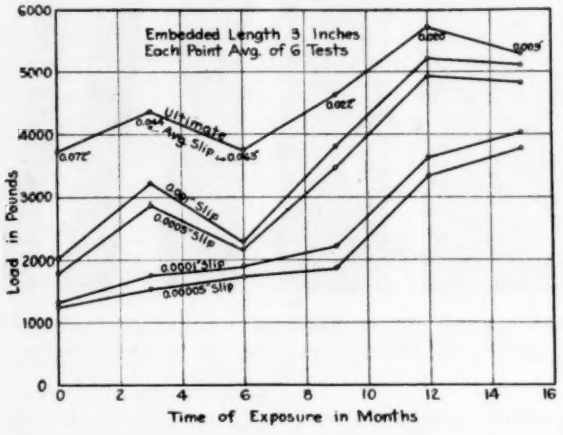
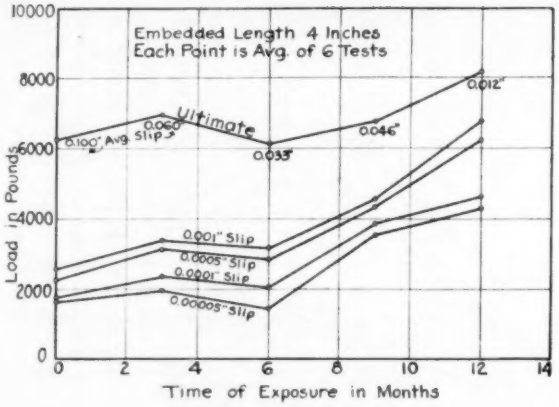


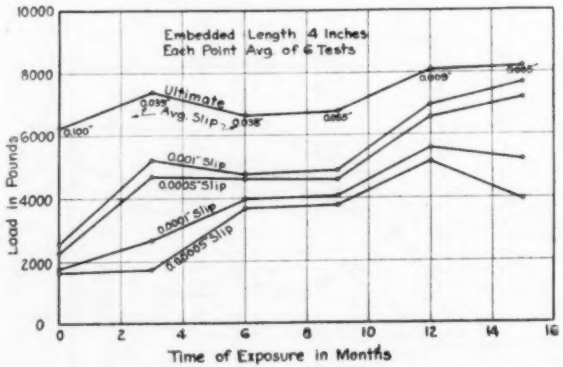
FIG. 8—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF $\frac{3}{8}$-IN. ϕ DEFORMED BARS

(See text, Part II)

than the nine months group. The coat at fifteen months was exceedingly loose and heavy.

The total load on the embedded bars at various stages of slip and at ultimate load are shown in Fig. 7 to 16 inclusive. The bond strength is progressively higher for the 9, 12, and 15 months, rusted bars as compared with the unrusted ones. These correspond to the more





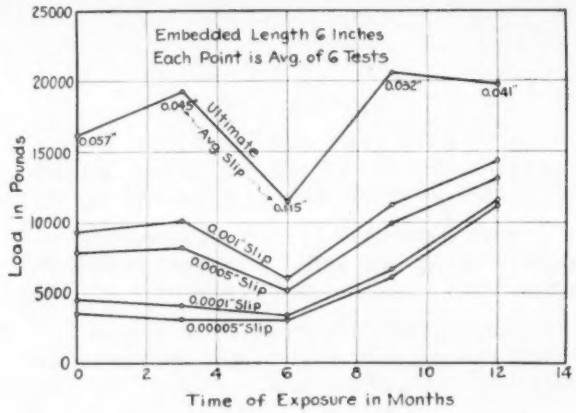
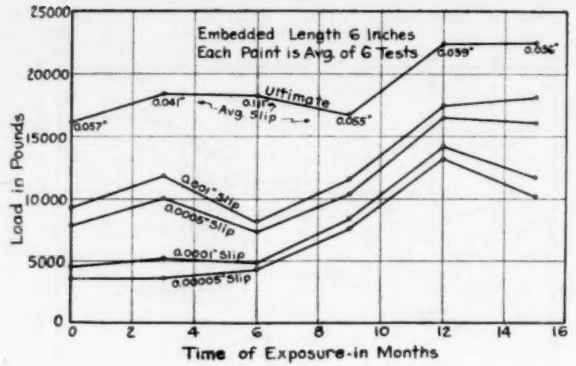


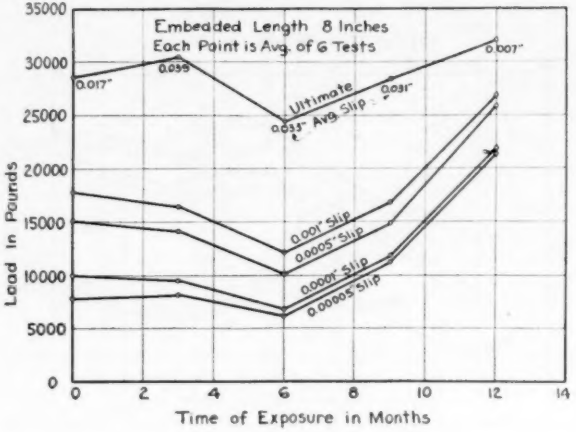
FIG. 9 (UPPER LEFT)—EFFECT OF PRIOR EXPOSURE IN MOIST ROOM ON 28 DAY BOND STRENGTH OF ½-IN. ϕ DEFORMED BARS

FIG. 10 (LOWER LEFT)—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF ½-IN. ϕ DEFORMED BARS

FIG. 11 (ABOVE)—EFFECT OF PRIOR EXPOSURE IN MOIST ROOM ON 28 DAY BOND STRENGTH OF ¾-IN. ϕ DEFORMED BARS

(See text, Part II)





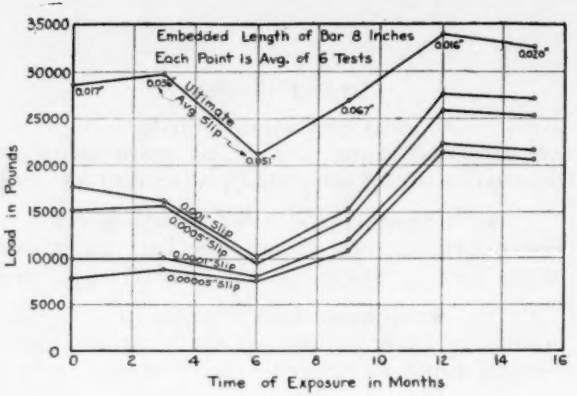


FIG. 12 (UPPER LEFT)—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF $\frac{3}{4}$-IN. DEFORMED BARS

FIG. 13 (LOWER LEFT)—EFFECT OF PRIOR EXPOSURE IN MOIST ROOM ON 28 DAY BOND STRENGTH OF 1-IN. □ DEFORMED BARS

FIG. 14 (ABOVE)—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF 1-IN. □ DEFORMED BARS

(See text, Part II)

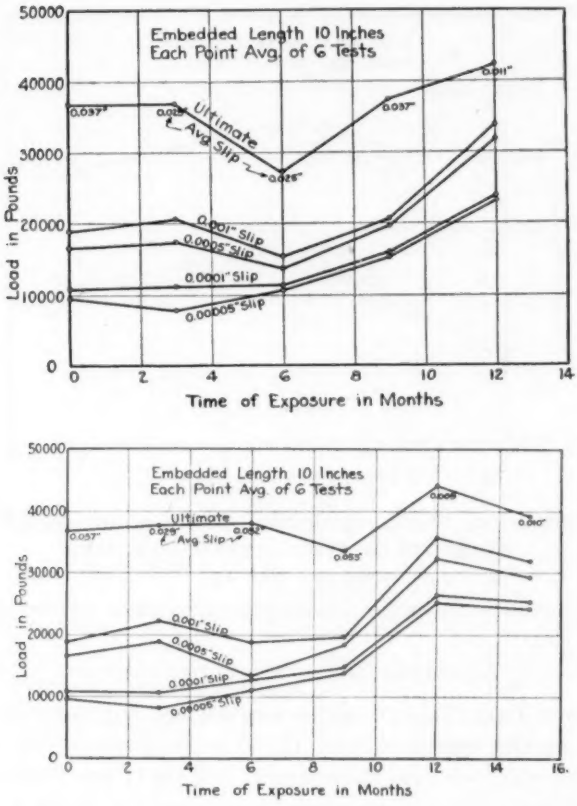


FIG. 15—EFFECT OF PRIOR EXPOSURE IN MOIST ROOM ON 28 DAY BOND STRENGTH OF 1¼-IN. □ DEFORMED BARS

FIG. 16—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF 1¼-IN. □ DEFORMED BARS

(See text, Part II)

advanced stages of loose, flaky rust. In the twelve and fifteen months groups part of the superiority may be attributed to the fact that the compressive strength of the concrete was somewhat above the average of all of the tests; but the total increase is greater than that which could be accounted for in this way. Three and six months of exposure seemed to have little effect upon the bond strength, particularly at slips of 0.00005 and 0.0001 in. For larger values of slip the six months tests were in some cases lower in bond strength than the unrusted bars. The general conclusion from these tests is, however, that advanced degrees of rust (provided there is no appreciable loss in

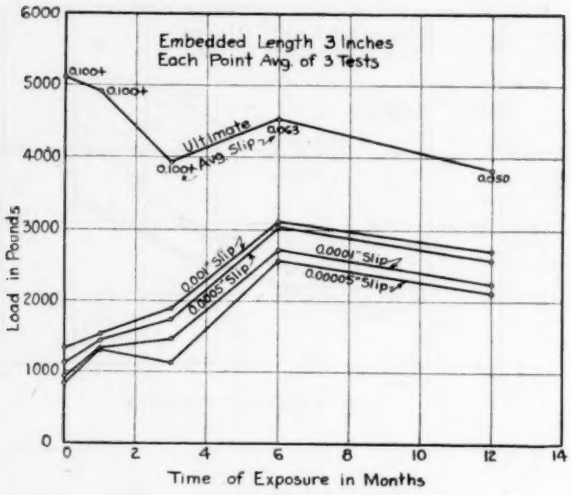


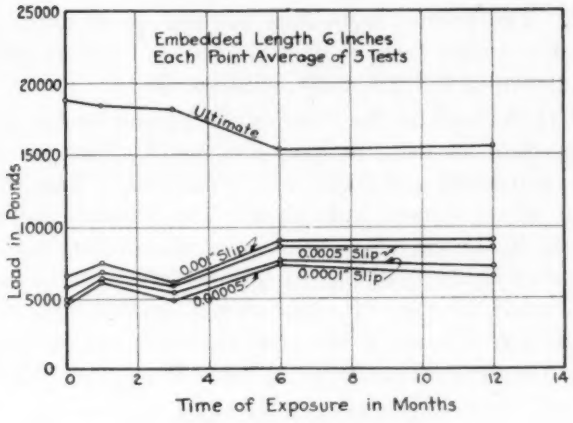
FIG. 17—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF $\frac{3}{8}$-IN. ϕ DEFORMED BARS
(See text, Part III)

net area) increases the pullout strength in bond of deformed reinforcing bars.

TEST PROGRAM AND RESULTS—PART III

The tests in Part III represent a series which supplement and serve as a check on the results of Part II. The concrete in this series was vibrated into place whereas the concrete in Part II was placed by hand-rodding. Three sizes of bar were used, namely, $\frac{3}{8}$-in.ϕ deformed, $\frac{3}{4}$-in.ϕ deformed and 1-in. square deformed. Three pullout specimens were made of each of the above bar sizes for outside exposures of 0, 1, 3, 6, and 12 months. The exposure started on Nov. 14, 1938. The concrete mix was the same as that used in Part II but 6 by 12-in. control cylinders were used with concrete vibrated into place. The average compressive strengths were 2320, 2410, 2300, 2390, and 2130 p.s.i. for the 0, 1, 3, 6, and 12-month groups with a general average of 2310 p.s.i. This average is 300 p.s.i. lower than the average of the rodded 3 by 6-in. cylinders used in Part II. The physical properties of the steel were given in Table 3.

The 6-in. diameter pullouts were filled completely and the $1\frac{1}{8}$-in. vibrator rod was held in each quadrant of the cross-section for five seconds, meaning that these specimens were vibrated for a total of twenty seconds. The 10-in. diameter specimens were made in a similar fashion except that the above procedure was carried out when the mold was half full and again when entirely filled. Three 6 by 12-in.



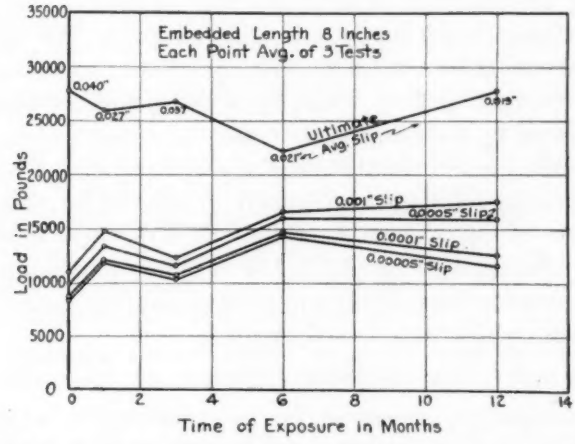


FIG. 18—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF $\frac{3}{4}$-IN. ϕ DEFORMED BARS

FIG. 19—EFFECT OF PRIOR EXPOSURE OUT OF DOORS ON 28 DAY BOND STRENGTH OF 1-IN. $\square$ DEFORMED BARS

(See text, Part III)

control cylinders were made for each mix in oiled steel molds. The molds were half-filled and the vibrator was applied for twenty seconds; then the molds were completely filled and the vibrator applied for another twenty seconds.

The appearance of the rusted bars in this series was quite similar to that of the bars in Part II at similar stages. The bars at twelve months were coated with a heavy rust coat which appeared to be somewhat more solid and not quite as flaky as the same stage of rust

in Part II. The pullout tests were carried out in exactly the same manner as the similar tests in Part II and the loads at various stages of slip are shown in Fig. 17 to 19 inclusive.

In Part III the load on the bars for the six and twelve months tests were higher than the load at zero months exposure for the various slips between 0.00005 and 0.001 in. for all sizes of bars but the load at ultimate failure showed a decrease. The ultimate strength of the concrete used in the twelve months series was slightly below average. This is reflected in the results of the pullout tests but the same general conclusions made for Part II apply as well to Part III. In Part III the six months results are better than the zero, one, or three months results for all stages of slip except ultimate load.

SUMMARY AND CONCLUSIONS

1. Bond pullout tests of deformed bars of the type tested usually showed higher strength in the rusted condition at low values of slip than corresponding tests in the unrusted condition.

2. The ultimate pullout strength of the deformed bars was not greatly affected by their condition of rust.

3. The effect upon bond pullout strength produced by brushing rusted reinforcing bars was inconclusive, producing increased strength at low values of slip in some cases and but little effect in other cases.

4. The total amount of slip before reaching maximum load was usually greater for bars in the unrusted or slightly rusted condition than for those which were heavily rusted.

ACKNOWLEDGMENT

This investigation, sponsored by the Concrete Reinforcing Steel Institute, was started in 1937 under the direction of Inge Lyse, formerly Research Professor of Engineering Materials at Lehigh University and now Professor of Reinforced Concrete and Solid Bridges at the Norges Tekniske Hoiskole at Trondheim, Norway. The investigation was a regular research project of the Fritz Engineering Laboratory, which is under the direction of Prof. Hale Sutherland, Head of the Department of Civil Engineering. Acknowledgment is due to Howard Godfrey, Engineer of Tests at the Laboratory, for his assistance in carrying out the work of the investigation.

Discussion, to close in February 1941 JOURNAL, *should reach A. C. I. Secretary in triplicate by Dec. 1, 1940.*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1940*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

How can a mixer drum be kept clean? (37-61)

Q—After a concrete mixer is used, part of the wet mixture adheres to the bottom and sides of the metal container. If this wet cement is not carefully washed out as soon as the mixer is shut down it will harden and become, in effect, a part of the mixer. Subsequent use of the mixer only makes matters worse as the new cement will adhere to the old. Sometimes it is possible to knock this old cement loose from the mixer by striking the metal on the outside with a heavy hammer. When this does not do the trick it is necessary to take a chisel and cut away the cement. This process is often disastrous because a hole might be made in the metal container. What can we do to loosen up this old cement without doing serious damage to the metal container and how is the condition to be prevented.

A—These few comments do not purport to be a "good" answer to the question. There is no *good* answer. In two cases of record, dynamite was proposed as a good answer, but in both instances the container was completely ruined. The only real answer is to send a man inside with a heavy hammer and suitable chisel, or a very small pavement breaker. Care must be taken to prevent denting the drum shell. For that reason, pounding on the outside is objectionable and should be avoided. With small drums that are not lined, pounding on the outside will sometimes dislodge a thin layer of concrete, but will not be effective for a large lump that has set up in the corner, or one that is reinforced by the blades or blade clips. As a detail of operation, pounding on the blades will help loosen up the concrete after some of it has been broken away by hammer and chisel.

Much the best advice is—do the cleaning as quickly as possible after the mixing operation has been finished for the day. Except for quick setting cement a few minutes work in the drum can leave it almost spotlessly clean which is of great importance as concrete has a much greater tendency to build on itself than on shiny metal. Frequently tumbling a few cubic feet of crushed stone in the drum, after washing out, is effective in removing small deposits.

In general, it is easier to keep a used mixer clean than a new one unless the drum shell has been damaged by pounding.

It would be unfair to leave this subject without pointing out that the way the cement is admitted to the mixer has much to do with concrete building up, and by correcting the charging sequence, or position of the cement stream as it enters the drum may eliminate entirely the cause of the concrete building up and prevent it. The main point involved is that unmixed cement should not be permitted to strike the damp surface of the drum shell, or the blades close to the drum. It should be insulated by a layer of aggregates or water, preferably aggregates. CHARLES F. BALL*

The building up of material in a concrete mixer is usually due primarily to the method which is followed in introducing the water. When the water is allowed to start into the drum two or three seconds prior to the entrance of the aggregate and the water pipe is turned to direct the flow toward the down turning side of the drum, there is opportunity for washing the buckets prior to contact with fresh material. In 90 per cent of the cases this procedure usually stops any build-up of concrete.

*Chief Engineer, Construction Equipment Division, Chain Belt Co., Milwaukee.

Another reason for this build-up is frequently in the fact that cement first enters the drum. If a change is made in the sequence of charging aggregates to the skip or batch hopper so that some sand or gravel enters the drum prior to the cement, a scouring action is thus obtained which helps to keep the drum clean.

Either one of these two procedures or a combination will, in most cases, eliminate any build-up of concrete in a mixer drum.

LION GARDINER*

How fast to place concrete dam sections and avoid shrinkage difficulties? (37-62)

Q—We are making up a tentative progress schedule for the construction of a dam and an important governing factor will be the setting time required for the mass concrete.

In the lower part of the dam the maximum size of an individual section will be 37 by 37 ft., and we propose bringing this up in lifts of 5 ft. Can you advise us the fastest rate that we can pour without running into trouble from shrinkage.

We know that in some of the larger dams which have been constructed in recent years it has been a matter requiring considerable study, and it occurs to us that if you have not available some specific information on the subject you might be able to obtain it from data pertaining to some recent projects.

A—It has been found that mass concrete blocks in which the maximum lateral dimensions are 40 ft. or less can be cast rapidly. Five foot lifts could probably be cast every two days without danger of cracking, provided reasonable care is taken to prevent the surface from cooling rapidly through a large range after removing the forms. However, if five-foot blocks are cast at as short an interval as two days for a considerable height, then consideration might be given to modification of the schedule to prevent the horizontal work joints from opening.

ROY W. CARLSON†

Tests for scouring of concrete by sand-laden water (37-63)

Q—We are in need of information as to methods of testing for the resistance of concrete to the scouring action of sand-laden water.

We also need information as to form lining to minimize pitted surfaces.

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*Vice-president Jaeger Machine Co., Columbus, Ohio.
†Associate Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

A—In Bulletin 1* of part 6, "Boulder Canyon Project Final Report," under the title, "Model Studies of Spillways," Chapter 6 is devoted to, "Erosion of Concrete Blocks under High Velocity." The data presented are based on erosion by clear water. In the tests reported streams or sheets of water have been projected against concrete specimens having various surface finishes.

Tests of absorptive form-lining as a means for reducing pitting on formed concrete surfaces and producing more durable and sightly surfaces are described in an article in *Engineering News Record*, Jan. 4, 1940, based on tests made by the Bureau of Reclamation.

*Bulletin obtainable from Bureau of Reclamation, Washington, D. C. at $1.00 in paper; $1.50 in cloth.

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER, 1940*

Proposed Building Regulations for Reinforced Concrete

Presented by A. C. I. Committee 318 (formerly 501) Standard Building Code

A. W. STEPHENS, CHAIRMAN; R. R. ZIPPRODT, SECRETARY

The present revision of the Building Regulations for Reinforced Concrete differs from the 1936 Tentative Building Regulations in the following respects:

1. Editorial changes have been made which place the notations applying to any particular Chapter, at the head of that Chapter, instead of in Chapter I.

2. There has been considerable revision of Chapter III—Concrete Quality and Working Stresses, to make it conform to the most recent developments in practice. Important changes have been made in Table 305(a)—Allowable Unit Stresses in Concrete, and in Section 306(b) relating to the size of reinforcement for, and the permissible span of one-way slabs.

3. In Chapter VII—Flexural Computations, material changes in the moment coefficients for the design of approximately equal spans, have been made in Section 701(c); also, in Section 706. Section 709 —Floors with Supports on Four Sides, has been revised, and considerably simplified for more ready use by the designing engineer. Tables of design factors for such slabs have been added as a footnote.

4. Chapter X—Flat Slabs, has been completely revised to take full advantage of the principles of continuity which are coming into general acceptance by designing engineers. The revision, however, still provides for design of flat slabs by the use of moment coefficients, under certain conditions which are fully outlined in Section 1003.

5. In Chapter XI—Reinforced Concrete Columns and Walls, Sections 1109 and 1110, governing the design of columns subjected to combined axial and bending stresses, have been materially changed.

6. Chapter XII—Footings, has been generally revised and rewritten, to conform to present practice in the design of isolated square and rectangular footings.

Intensive work was done by the Committee following discussion of proposed code revisions at the 1940 convention, and on June 22 all changes were referred to letter ballot of Committee 318 (formerly 501) which consists of 27 members; 24 members returned their ballots of whom 23 voted affirmatively and one negatively. Subsequently the report was released by the Standards Committee for presentation to the 1941 Convention on a motion for adoption.

The report is published here as the basis for discussion and convention action under paragraph (*j*), "Conduct of Committee Work," as follows:

(*j*) When a standard is formally presented to the Institute with the approval of four or more members of the Standards Committee, it shall be open for discussion on the floor of a convention. If approved by a majority vote of members present it shall be submitted, by reference, to letter ballot of the Institute membership and the standard shall become effective upon ratification by two thirds of those voting. An amendment to a proposed standard presented on the floor of the convention shall be sustained only when approved by nine tenths of the members voting and as so amended shall be submitted to letter ballot as provided above.

The membership of Committee 318, Standard Building Code, is as follows:

A. W. STEPHENS, Chairman
R. R. ZIPPRODT, Secretary

PAUL ANDERSEN	A. BURTON COHEN	T. D. MYLREA
R. L. BERTIN	J. DI STASIO	N. M. NEWMARK
H. P. BIGLER	ABRAHAM EPSTEIN	JOHN R. NICHOLS
A. J. BOASE	W. H. GRAVELL	L. H. NISHKIAN
R. D. BRADBURY	VERNON P. JENSEN	J. M. OXLEY
MILES N. CLAIR	N. D. MITCHELL	DEAN PEABODY, JR.
F. E. RICHART	HALE SUTHERLAND	C. A. WILLSON
W. C. SPIKER	L. C. URQUHART	R. B. YOUNG
	W. H. WHEELER	

Proposed

Building Regulations for Reinforced Concrete

Chapter I—General

101—Scope

(*a*) These regulations cover the use of reinforced concrete and plain concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of these materials. In all matters pertaining to design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102—Permits and Drawings

(*a*) Drawings and typical details of all reinforced concrete construction showing the size and position of all structural members, metal reinforcement, design strength of concrete, and the live load used in the design shall be filed with the building department as a permanent record before a permit to construct such work will be issued. All plans submitted for approval or use on the work shall clearly show the strength of concrete at a specified age for which all parts of the structure were designed. Calculations pertaining to the design shall be filed with the drawings when required by the Commissioner of Buildings.

103—Special Systems of Reinforced Concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by these regulations shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

104—Definitions

(*a*) The following terms are defined for use in this code:

Aggregate—Inert material which is mixed with portland cement and water to produce concrete.

Column—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip—A portion of a flat slab panel one-half panel in width consisting of the two adjacent quarter-panels on either side of the column center lines and extending through the panel in the direction of the span considered for bending.

Combination Column—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column—A column in which a steel or cast-iron section is completely encased in concrete containing spiral and longitudinal reinforcement.

Concrete—A mixture of portland cement, fine aggregate, coarse aggregate and water.

Deformed Bar—Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 W. & M. gage may be rated as a deformed bar.

Diagonal Band—A group of reinforcing bars covering a width approximately 0.4 the average span, placed symmetrically with respect to the diagonal running from corner to corner of the panel of a flat slab.

Direct Band—A group of reinforcing bars, covering a width approximately 0.4 of l_1, placed symmetrically with respect to the center lines of the supporting columns of a flat slab.

Drop Panel—The structural portion of a flat slab which is thickened in the area surrounding the column capital.

Effective Area of Concrete—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the flexural member.

Effective Area of Reinforcement—The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Flat Slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting columns.

Middle Strip—A portion of a flat slab panel one-half panel in width, symmetrical about the panel center line and extending through the panel in the direction of the span considered for bending.

Paneled Ceiling—A flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length—The distance along a panel side from center to center of columns of a flat slab.

Pedestal—An upright compression member whose height does not exceed three times its least lateral dimension.

Plain Concrete—Concrete without reinforcement, or reinforced only for shrinkage or temperature changes.

Ratio of Reinforcement—The ratio of the effective area of the reinforcement to the effective area of the concrete at any section of a flexural member.

Reinforced Concrete—Concrete in which reinforcement other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

Surface Water—The water carried by the aggregate except that held by absorption within the aggregate particles themselves.

Chapter II—Materials and Tests

200—Notation

D = Deflection of a floor member under load test.
L = Span of member under load test (See Section 202).
t = The total thickness or depth of a member under load test.

201—Tests

(*a*) The Commissioner of Buildings, or his authorized representative shall have the right to order the test of any material entering into concrete or reinforced concrete when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(*b*) Tests of materials and of concrete shall be made in accordance with the requirements of the American Society for Testing Materials as noted elsewhere in this chapter. The complete records of such tests shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work, and shall be preserved by the engineer or architect for two years after the completion of the structure.

202—Load Tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given in the following:

$$D = \frac{.001\,L^2}{12t} \qquad (1)$$

all terms expressed in the same units.

If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the slabs or beams show a recovery of at least seventy-five per cent of the observed deflection.

203—Supervision

(*a*) All concrete work shall be supervised by the architect or engineer responsible for its design, or by a competent representative responsible to the architect or engineer. A record shall be kept of such supervision, which record shall cover the quality and quantity of concrete materials, the mixing and placing of the concrete, and the placing of the reinforcing steel. A complete record shall also be kept of the progress of the work and of the temperatures, when these fall below 40 degrees F., and of the protection given to the concrete while curing. This record shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work and shall be preserved by the architect or engineer for two years after the completion of the work.

204—Portland Cement

(*a*) Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Serial Designation: C9-38) or the "Standard Specifications for High-Early Strength Portland Cement" (A.S.T.M. Serial Designation: C74-39).

205—Concrete Aggregates

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (A. S. T. M. Serial Designation: C33-39), provided however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness, fire-resistance, and wearing qualities may be used under Section 302(a) Method 2, where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between sides of the forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars.

206—Water

(*a*) Water used in mixing concrete shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials, or other deleterious substances.

207—Metal Reinforcement

(*a*) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A15-39), or for "Rail-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A16-35), or for "Axle-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A160-39).

(*b*) Cold-drawn wire or welded wire fabric for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (A. S. T. M. Serial Designation: A82-34), or for Welded Steel Wire Fabric for Concrete Reinforcement (A. S. T. M. Serial Designation: A185-37).

(*c*) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Bridges and Buildings" (A. S. T. M. Serial Designation: A7-39).

(*d*) Cast-iron sections for composite columns shall conform to the "Tentative Specifications for Cast Iron Pit-cast Pipe for Water and Other Liquids" (A. S. T. M. Serial Designation: A44-39T).

208—Storage of Materials

(*a*) Cement and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used for concrete.

Chapter III—Concrete Quality and Working Stresses

300—Notation

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f_c = Compressive unit stress in extreme fiber of concrete in flexure.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

f_r = Allowable unit stress in the metal core of a composite column.

f'_r = Allowable unit stress on unencased steel columns and pipe columns.

f_s = Tensile unit stress in longitudinal reinforcement; nominal working stress in vertical column reinforcement.

f_v = Tensile unit stress in web reinforcement.

n = Ratio of modulus of elasticity of steel to that of concrete, assumed in Section 305(a) as equal to $\frac{30{,}000}{f'_c}$.

r_a = Allowable unit working stress in concrete over the loaded area of a pedestal, pier or footing.

u = Bond stress per unit of surface area of bar.

v = Shearing unit stress.

v_c = Shearing unit stress permitted on the concrete of the web.

301—Concrete Quality

(*a*) For the design of reinforced concrete structures, the value of f'_c used for determining the working stresses as stipulated in Section 305 shall be based on the specified minimum ultimate 28-day compressive strength of the concrete, or on the specified minimum ultimate compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) All concrete exposed to the action of the weather shall have a water-content of not to exceed six gallons per sack of cement.*

*In climates where frost action is not severe this section should be omitted.

302—Determination of Strength-Quality of Materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

TABLE 302(*a*)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Content U. S. Gallons Per 94-lb. Sack of Cement	Assumed Compressive Strength at 28 Days—p.s.i.
7½	2000
6¾	2500
6	3000
5	3750

NOTE—In interpreting this table, surface water contained in the aggregate must be included as part of the mixing water in computing the water-content.

Method 2—Controlled Concrete:

Proportions of the materials and water-content, other than those shown in Table 302(*a*) may be used provided that the strength-quality of the materials proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A.S.T.M. Serial Designation: C39-39). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The water-content used in the concrete for the structure as determined from the curve, shall correspond to a strength which is fifteen per cent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

303—Tests on Concrete

(*a*) The Commissioner of Buildings shall require a reasonable number of compression tests to be made during the progress of the work. Such tests shall be made in accordance with the "Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field" (A. S. T. M. Serial Designation C31-39), and cured in accordance with the requirements for laboratory control tests.

(*b*) Not less than three specimens shall be made for each test; nor less than one test for each 250 cu. yd. of concrete.

(*c*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths

of the concrete is established by test for the materials and proportions used.

(*d*) Where the average strength of the laboratory control cylinders for any portion of the structure falls below the minimum ultimate compressive strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mixture or in the water-content for the remaining portion of the structure. In cases where the average strength of the cylinders cured on the job falls below the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength and may require load tests to be made on the portions of the building so affected as specified in Section 202.

304—Concrete Proportions and Consistency

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than thirty per cent nor greater than fifty per cent of the total, except that these proportions do not necessarily apply to light-weight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for Ready Mixed Concrete shall conform to the "Standard Specifications for Ready Mixed Concrete" (A. S. T. M. Serial Designation: C94-38).

305—Allowable Unit Stresses in Concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(a) where f'_c equals the minimum specified ultimate compressive strength at 28 days.

306—Allowable Unit Stresses in Reinforcement

Unless otherwise provided in these Regulations, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)

*Wherever practicable such measurement shall be by weight rather than by volume.

TABLE 305(*a*)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable Unit Stresses				
		For Any Strength of Concrete as Fixed by Test in Accordance with Section 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302			
			f'_c = 2000 p.s.i. n = 15	f'_c = 2500 p.s.i. n = 12	f'_c = 3000 p.s.i. n = 10	f'_c = 3750 p.s.i. n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c	$0.03f'_c$ but not to exceed	60 75 p.s.i.	75	75	75
‡Bond: u						
In beams and slabs and one-way footings:						
Plain bars	u	$0.04f'_c$ but not to exceed	80 160 p.s.i.	100	120	150
Deformed bars	u	$0.05f'_c$ but not to exceed	100 200 p.s.i.	125	150	188
In two-way footings:						
Plain bars (Hooked)	u	$0.045f'_c$ but not to exceed	90 160 p.s.i.	113	135	160
Deformed bars (Hooked)	u	$0.056f'_c$ but not to exceed	112 200 p.s.i.	140	168	200
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area or less†	f_c	$0.375f'_c$	750	938	1125	1405

*See Section 807. **See Section 905(*a*) and 808(*a*).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Section 903(*a*)), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 p.s.i. for plain bars and 250 p.s.i. for deformed bars. The values given for two-way footings include an allowance for special anchorage.

and (f_v = Tensile unit stress in web reinforcement)

20,000 p.s.i. for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

18,000 p.s.i. for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

(*b*) *Tension in One-Way Slabs of Not More Than 12 Feet Span*

For the main reinforcement, $\frac{3}{8}$ inch or less in diameter, in one-way slabs, 50 per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(*c*) *Compression, Vertical Column Reinforcement*

(f_s = Nominal working stress in vertical column reinforcement).

Forty per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i. (See Section 1103(a). For requirements governing column spirals, see Section 1103(d).

(f_r = Allowable unit stress in the metal core of composite and combination columns):

Structural steel sections	16,000 p.s.i.
Cast iron sections	10,000 p.s.i.
Steel pipe	See limitations of Section 1106(b)

(*d*) For compression reinforcement in flexural members see Section 706(b).

Chapter IV—Mixing and Placing Concrete

401—Preparation of Equipment and Place of Deposit

(*a*) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the spaces to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and masonry filler units that will be in contact with concrete shall be well drenched (except in freezing weather), and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(*b*) Water shall be removed from place of deposit before concrete is placed unless otherwise permitted by the Commissioner of Buildings.

402—Mixing of Concrete

(*a*) Unless otherwise authorized by the Commissioner of Buildings, the mixing of concrete shall be done in a batch mixer of approved type.

(*b*) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(*c*) For job mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer. A longer mixing period may be required for mixers larger than one cubic yard capacity.

(*d*) Ready mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the "Standard Specifications for Ready Mixed Concrete" (A. S. T. M. Serial Designation C94-38).

403—Conveying

(*a*) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

(*b*) Equipment for chuting, pumping and pneumatically conveying concrete shall be of such size and design as to insure a practically continuous flow of concrete at the delivery end without separation of the materials.

404—Depositing

(*a*) Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. The

concreting shall be carried on at such a rate that the concrete is at all times plastic and flows readily into the spaces between the bars. No concrete that has partially hardened or been contaminated by foreign materials shall be deposited on the work, nor shall retempered concrete be used.

(*b*) When concreting is once started, it shall be carried on as a continuous operation until the placing of the panel or section is completed. The top surface shall be generally level. When construction joints are necessary, they shall be made in accordance with Section 508.

(*c*) All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around the reinforcement, embedded fixtures and into the corners of the forms. Vibrators may be used to aid in the placement of the concrete provided they are used under experienced supervision, and the forms are designed to withstand their action.

(*d*) Where conditions make compacting difficult, or where the reinforcement is congested, batches of mortar containing the same proportions of cement to sand as used in the concrete, shall first be deposited in the forms to a depth of one inch.

405—Curing

(*a*) In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days.

406—Cold Weather Requirements

(*a*) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(*b*) All concrete materials and all reinforcement, forms, fillers and ground with which the concrete is to come in contact, shall be free from frost. Whenever the temperature of the surrounding air is below 40 degrees Fahrenheit, all concrete when placed in the forms shall have a temperature of between 60 and 90 degrees Fahrenheit and shall be maintained at a temperature of not less than 50 degrees Fahrenheit for at least 72 hours for normal concrete or 24 hours for high-early-strength concrete, or for as much more time as is

necessary to insure proper curing of the concrete. The housing, covering or other protection shall remain in place and intact at least twenty-four hours after the artificial heating is discontinued. No dependence shall be placed on salt or other chemicals for the prevention of freezing. Manure, when used for protection, shall not be applied directly to the concrete.

Chapter V—Forms and Details of Construction

501—Design of Forms

(*a*) Forms shall conform to the shape, lines, and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

502—Removal of Forms

(*a*) Forms shall be removed in such manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, the removable floor forms, beam and girder sides, column and similar vertical forms may be removed after twenty-four hours, providing the concrete is sufficiently hard not to be injured thereby. In no case shall the supporting forms or shoring be removed until the members have acquired sufficient strength to support safely their weight and the load thereon. The results of suitable control tests may be used as evidence that the concrete has attained such sufficient strength.

503—Pipes, Conduits, etc., Embedded in Concrete

(*a*) Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete except in passing through from one side to the other of a floor, wall or beam. Electric conduits and other pipes whose embedment is allowed shall not, with their fittings, displace that concrete of a column on which stress is calculated or which is required for fire protection, to greater extent than four per cent of the area of the cross section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced concrete, provided they are not exposed to rusting or other deterioration, are of uncoated iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over two inches, and are spaced not less than three diameters on centers. Embedded pipes

or conduits other than those merely passing through, shall not be larger in outside diameter than one-third the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than three diameters on centers, nor so located as unduly to impair the strength of the construction. Circular uncoated or galvanized electric conduit of iron or steel may be considered as replacing the displaced concrete.

504—Cleaning and Bending Reinforcement

(*a*) Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for stirrups and ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. Bends for other bars shall be made around a pin having a diameter not less than six times the minimum thickness of the bar, except that for bars larger than one inch, the pin shall be not less than eight times the minimum thickness of the bar. All bars shall be bent cold.

505—Placing Reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs and spacers. The minimum clear distance between parallel bars shall be one and one-half times the diameter for round bars and twice the side dimension for square bars. If special anchorage as required in Section 903 is provided, the minimum clear distance between parallel bars shall be equal to the diameter for round bars and one and one-half times the side dimension for square bars. In no case shall the clear distance between bars be less than one in., nor less than one and one-third times the maximum size of the coarse aggregate.

(*b*) When wire or other reinforcement, not exceeding one-fourth inch in diameter is used as reinforcement for slabs not exceeding ten feet in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at mid-span; provided such reinforcement is either continuous over, or securely anchored to the support.

506—Splices and Offsets in Reinforcement

(*a*) In slabs, beams and girders, splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the minimum spacing of bars shall be as specified in Section 505.

(*b*) Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion shall not be more than 1 in 6, and in the case of tied columns the ties shall be spaced not over three inches on centers for a distance of one foot below the actual point of offset.

507—Concrete Protection for Reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than $\frac{5}{8}$ inch in diameter and one and one-half inches for bars $\frac{5}{8}$ inch or less in diameter.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of metal reinforcement shall be at least three-fourths inch.

(*c*) If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

508—Construction Joints

(*a*) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a joint is to be made, the surface of the concrete shall be thoroughly cleaned and all laitance removed. In addition to the foregoing, vertical joints shall be thoroughly wetted but not saturated, and slushed with a coat of neat cement grout immediately before placing of new concrete.

(*b*) At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(*c*) Construction joints in floors shall be located near the middle of the spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

Chapter VI—Design—General Considerations

600—Notation

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

n = Ratio of modulus of elasticity of steel to that of concrete = $\frac{E_s}{E_c}$; assumed as equal to $\frac{30{,}000}{f'_c}$.

601—Assumptions

(*a*) The design of reinforced concrete members shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending. The following assumptions shall be made:

1. The steel takes all the tensile stress.
2. In determining the ratio n for design purposes, the modulus of elasticity for the concrete shall be assumed as $1000\,f'_c$, and that for steel as 30,000,000 p.s.i.

602—Design Loads

(*a*) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

603—Resistance to Wind Forces

(*a*) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(*b*) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead- and live-loads.

(*c*) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the allowable stresses for combined live- and dead-loads provided in Sections 305, 306 and 1110 by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

Chapter VII—Flexural Computations

700—Notation

A = Span length between opposite supports in one direction.

B = Span length at right angles to A.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the diameter of a round bar or side of a square bar.

e_A = Factor modifying r_A, used in obtaining an equivalent uniform load for bending moments on span A.

e_B = Factor modifying r_B, used in obtaining an equivalent uniform load for bending moments on span B.

E = The modulus of elasticity of concrete in compression.

F_AA = The distance between lines of inflection in span A, considering span A only to be loaded.

F_BB = The distance between lines of inflection in span B, considering span B only to be loaded.

F_A = Ratio of the distance between assumed inflection points of the span A to span A in an isolated strip extending the entire width of the structure when a uniformly distributed load is applied to span A only.

F_B = Ratio as defined above, but applying to span B.

h = Unsupported length of a column.

I = Moment of inertia of a section about the neutral axis for bending.

K = The stiffness factor, that is, the moment of inertia divided by the span.

K_A = Stiffness factor $\frac{I}{A}$ for span A of panel AB.

K_B = Stiffness factor $\frac{I}{B}$ for span B of panel AB.

K_{AR} = Stiffness factors for any span adjacent to and continuous with span A.

K_{BR} = Stiffness factors for any span adjacent to and continuous with span B.

l = Span length of slab or beam.

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment (See Section 701).

N = The sum of the lengths of those edges of panel AB which are also edges of adjacent panels continuous with AB.

$q_A = 6r_A\ (1 - e_A)$.

$q_B = 6r_B\ (1 - e_B)$.

r_A = Proportion of the total load carried by span A of slab.

r_B = Proportion of the total load carried by span B of slab.

t_1 = Minimum total thickness of slab.

w = Uniformly distributed load per unit of length of beam or slab.

x = Distance from face of support to point in span.

701—General Requirements

(*a*) All members of frames or continuous construction shall be designed to resist at all sections the maximum moments and shears produced by dead load, live load and wind load, as determined by the theory of elastic frames in which the simplified assumptions of Section 702 may be used.

(*b*) Approximate methods of frame analysis are satisfactory for buildings of usual types of construction, spans and story heights.

(*c*) In the case of two or more approximately equal spans (the larger of two adjacent spans not exceeding the shorter by more than 20 per cent) with loads uniformly distributed, where the unit live load does not exceed three times the unit dead load, design for the following moments and shears is satisfactory:

Positive moment at center of span

End spans ..... $\frac{1}{14}\ wl'^2$

Interior spans ..... $\frac{1}{16}\ wl'^2$

Negative moment at exterior face of first interior support

Two spans ..... $\frac{1}{9}\ wl'^2$

More than two spans ..... $\frac{1}{10}\ wl'^2$

Negative moment at other faces of interior supports

$$\frac{1}{11} wl'^2$$

Negative moment at face of all supports for, (a) slabs with spans not exceeding ten feet, and (b) beams and girders where ratio of sum of column stiffnesses to beam stiffness exceeds eight

$$\frac{1}{12} wl'^2$$

Shear in end members at first interior support $1.15 \frac{wl'}{2}$

Shear at other supports $\frac{wl'}{2}$

702—Conditions of Design*

(*a*) *Arrangement of Live Load*

1. The live load may be considered to be applied only to the floor under consideration, and the far ends of the columns may be assumed as fixed.

2. Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans.

(*b*) *Span length*

1. The span length, l, of members that are not built integrally with their supports shall be the clear span plus the depth of the slab or beam but shall not exceed the distance between centers of supports.

2. In analysis of continuous frames, center to center distances, l and h, may be used in the determination of moments. Moments at faces of supports may be used for design of beams and girders.

3. Solid or ribbed slabs with clear spans of not more than ten feet that are built integrally with their supports may be designed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and the width of beams otherwise neglected.

(*c*) *Stiffness*

1. The stiffness, K, of a member is defined as EI divided by l or h.

2. In computing the value of I of slabs, beams, girders, and columns, the reinforcement may be neglected. In T-shaped sections allowance shall be made for the effect of flange.

*Chapter VII deals with floor members only. For moments in columns see Section 1108.

3. Any reasonable assumption may be adopted as to relative stiffness of columns and of floor system. The assumption made shall be consistent throughout the analysis.

(*d*) *Haunched Floor Members*

1. When members are widened near the supports, the additional width may be neglected in computing moments, but may be considered as resisting the resulting moments and shears.

2. When members are deepened near the supports, they may be analyzed as members of constant depth provided the minimum depth only is considered as resisting the resulting moments; otherwise an analysis taking into account the variation in depth is required. In any case, the actual depth may be considered as resisting shear.

(*e*) *Limitations*

1. Wherever at any section positive reinforcement is indicated by analysis, the amount provided shall be not less than .005 $b'd$ except in slabs of uniform thickness.

2. Not less than 0.005 $b'd$ of negative reinforcement shall be provided at the outer end of all members built integrally with their supports.

3. Where analysis indicates negative reinforcement along the full length of a span, the reinforcement need not be extended beyond the point where the required amount is 0.0025 $b'd$ or less.

4. In slabs of uniform thickness the minimum amount of reinforcement in the direction of the span shall be:

For structural, intermediate and hard grades and rail steel . . . . . 0.0025 bd

For steel having a minimum yield point of 56,000 p. s. i. . . . . 0.002 bd

703—Depth of Beam or Slab

(*a*) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face of the structural members. Any floor finish not placed monolithically with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithically with the structural slab in buildings of the warehouse or industrial class, there shall be placed an additional depth of one-half inch over that required by the design of the member.

704—Distance between Lateral Supports

(*a*) The clear distance between lateral supports of a beam shall not exceed thirty-two times the least width of compression flange.

705—Requirements for T-Beams

(*a*) In T-beam construction the slab and beam shall be built integrally or otherwise effectively bonded together. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(*b*) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(*c*) Where the principal reinforcement in a slab ,which is considered as the flange of a T-beam (not a joist in concrete joist floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, nor in any case eighteen inches.

(*d*) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Section 505 relating to the spacing of bars, and 404(d), relating to the placing of concrete shall be fully met.

(*e*) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(*f*) Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

706—Compression Steel in Flexural Members

(*a*) Compression steel in beams, girders, or slabs shall be anchored by ties or stirrups not less than ¼ inch in diameter spaced not farther apart than 16 bar diameters, or 48 tie diameters. Such stirrups or ties shall be used throughout the distance where the compression steel is required.

(*b*) The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from the calculations assuming a straight-line relation between stress and strain and the modular ratio given in Section 601, but not of greater value than the allowable stress in tension.

707—Shrinkage and Temperature Reinforcement

(*a*) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area *bd*, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than eighteen inches:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0022

708—Concrete Joist Floor Construction

(*a*) Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than thirty inches face to face. The ribs shall be straight, not less than four inches wide, nor of a depth more than three times the width.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the joists are used, and the fillers are so placed that the joints in alternate rows are staggered, the vertical shells of the fillers in contact with the joists may be included in the calculations involving shear or negative bending moment. No other portion of the fillers may be included in the design calculations.

(*c*) The concrete slab over the fillers shall be not less than one and one-half inches in thickness, nor less in thickness than one-twelfth of the clear distance between joists. Shrinkage reinforcement in the slab shall be provided as required in Section 707.

(*d*) Where removable forms or fillers not complying with (*b*) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between joists and in no case less than two inches. Such slab shall be reinforced at right angles to the joists with a minimum of .049 sq. in. of reinforcing steel per foot of width, and in slabs on which the prescribed live load does not exceed fifty lb. per sq. ft., no additional reinforcement will be required.

(*e*) When the finish used as a wearing surface is placed monolithically with the structural slab in buildings of the warehouse or industrial class, the thickness of the concrete over the fillers shall be one-half inch greater than the thickness used for design purposes.

(*f*) Where the slab contains conduits or pipes, the thickness shall not be less than one inch plus the total over-all depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to impair the strength of the construction.

709—Floors with Supports on Four Sides [1] [2]

(*a*) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with burned clay or concrete tile fillers, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The supports for the floor slabs may be walls, reinforced concrete beams, or steel beams fully encased in concrete.

Footnotes:

(1) For comparative use the moment of inertia of a slab shall be taken as that of the total plain concrete section.

(2) Formulas for F_A, F_B, e_A, e_B, r_A, r_B. (See "Slabs Supported on Four Sides" by J. DiStasio and R. P. Van Buren, JOURNAL of the A. C. I., January-February, 1936).

End Span, continuous at one end only

$$F_A = 1 - \frac{0.25}{1 + \frac{7K_A}{8K_{AR}}} \qquad (7)$$

Interior continuous span with K_{AR} the same for both adjacent spans continuous with A

$$F_A = \sqrt{1 - \frac{1}{1.5 + \frac{7K_A}{8K_{AR}}}} \qquad (8)$$

For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AR} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AR} for the span at the other end.

To obtain F_B replace K_A with K_B and K_{AR} with K_{BR}.

$$F_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)^3} = 1 - r_B \qquad (9)$$

$$e_A = \frac{2}{4 - \frac{F_B B}{F_A A}} \qquad (10a) \qquad e_B = \frac{2}{4 - \frac{F_A A}{F_B B}} \qquad (10b)$$

$e_A = 1.0$ for $\frac{F_B B}{F_A A} = 2$ $\qquad$ $e_B = 0.5$, as $\frac{F_A A}{F_B B} = 0$

The total load carried by a strip of slab of unit width, span A ,equals $r_A w A$ and is considered to vary in intensity from $r_A w\ (3e_A - 2)$ at the center of the span, to $r_A w\ (4 - 3\ e_A)$ at the supports.

The total load carried by a beam of span A, one-half panel tributary width, equals

$$1\text{-}r_A \frac{wBA}{2} \qquad (11)$$

and varies uniformly in intensity from $(1 + 2r_A - 3e_A r_A)\ \frac{wB}{2}$ at the center of the span to

$(1 - 4r_A + 3e_A r_A)\ \frac{wB}{2}$ at the supports.

When considering the B spans use the above expressions, replacing A with B, B with A, r_A with r_B, and e_A with e_B.

(*Footnote (2) continued next page*)

(*b*) *Minimum Slab Thickness*

The slab thickness shall satisfy prescribed working stresses and shall be not less than

$$t_1 = \frac{A + B - 0.10N}{72} \quad \text{.....(2)}$$

(*c*) *Bending Moments and Shears*

The bending moment at any section shall be determined with the same coefficients prescribed for one-way construction (Section 701(c), using the following equivalent uniform load per unit length of span considered:

Slab: Strip of unit width, span A, $(e_A r_A)w$ .....(3)

Beam: Span A, carrying one half of load from panel width B,

(*Footnote (2) continued from previous page.*

TABLE 1—F_A AND F_B

The values given in the table are for F_A directly. They are also the values for F_B when the designation K_A/K_{AB} is replaced by K_B/K_{BB}.

Span A	$\frac{K_A}{K_{AB}}$ =	0.00	0.25	0.50	0.67	0.80	1.00	1.25	1.50	2.00	4.00	0. ∞
Interior*.	F_A =	0.58	0.65	0.69	0.72	0.74	0.76	0.78	0.80	0.83	0.89	1.00
End.....	F_A =	0.75	0.80	0.83	0.84	0.85	0.87	0.88	0.89	0.91	0.95	1.00
Simple...	F_A =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

*For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AB} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AB} for the span at the other end.

For values of K_A/K_{AB} between 2/3 and 3/2 the values of F_A may be taken as 0.76 for interior spans and 0.87 for end spans.

TABLE 2

The value of e_A or e_B shall be taken as unity for the computation of shear and bending moment in slabs and beams where the span in direction under consideration is not rigidly attached to the supports at one or both ends of the span.

$\frac{F_A A}{F_B B}$	r_A or $1 - r_B$	e_A	$e_A r_A$	$1 - e_A r_A$	$q_A = 6 r_A (1 - e_A)$
0.00	1.00	1.00	1.00	0.00	0.00
0.50	0.89	1.00	0.89	0.11	0.00
0.55	0.86	0.92	0.79	0.21	0.41
0.60	0.82	0.86	0.71	0.29	0.69
0.65	0.78	0.81	0.63	0.37	0.89
0.70	0.74	0.78	0.58	0.42	0.98
0.80	0.66	0.73	0.48	0.52	1.07
0.90	0.58	0.69	0.40	0.60	1.08
1.00	0.50	0.67	0.33	0.67	1.00
1.10	0.43	0.65	0.28	0.72	0.90
1.20	0.37	0.63	0.23	0.77	0.82
1.30	0.31	0.62	0.19	0.81	0.71
1.40	0.27	0.61	0.16	0.84	0.63
1.50	0.23	0.60	0.14	0.86	0.55
1.60	0.20	0.59	0.12	0.88	0.49
1.80	0.15	0.58	0.09	0.91	0.38
2.00	0.11	0.57	0.06	0.94	0.28
3.00	0.04	0.55	0.02	0.98	0.11
$\frac{F_B B}{F_A A}$	r_B or $1 - r_A$	e_B	$e_B r_B$	$1 - e_B r_B$	$6 r_B (1 - e_B) = q_B$

$$\frac{B}{2}(1 - e_A r_A)w \quad \ldots\ldots (4)$$

The shear at any section at a distance x from the face of the support shall be taken as:

Slab: Strip of unit width, span A,

$$\left(\frac{A}{2} - x\right)\left(r_A - \frac{q_A{}^x}{A}\right)w \quad \ldots\ldots (5)$$

Beam: Span A, carrying one half of load from panel, width B,

$$\frac{B}{2}\left(\frac{A}{2} - x\right)\left(1 - r_A + \frac{q_A{}^x}{A}\right)w \quad \ldots\ldots (6)$$

For span B, use the above expressions substituting A for B, B for A, e_B for e_A, r_A for r_B, and q_B for q_A.

The factors e_A, r_A, etc., may be taken from Table 2, footnote (2) below after the ratio $F_A A/F_B B$ or $F_B B/F_A A$ on which they depend has been determined by the aid of Table 1, footnote (2); or the several factors may be computed from the formulas which appear in the footnote (2).

(*d*) *Arrangement of Reinforcement*

1. In any panel, the reinforcement per unit width in the long direction shall be at least one-third of that provided in the short direction.

2. The positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced twenty-five per cent.

3. At a non-continuous edge negative moment reinforcement per unit width in amount at least as great as one-half of that required for maximum positive moment for the center one-half of the panel shall be provided across the entire width of the exterior support.

4. The spacing of the reinforcement shall be not more than three times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

710—Maximum Spacing of Principal Slab Reinforcement

(*a*) In slabs other than concrete joist floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 707(a).

Chapter VIII—Shear and Diagonal Tension

800—Notation

A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

α = Angle between inclined web bars and axis of beam.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

f_v = Tensile unit stress in web reinforcement.

j = Ratio of distance between centroid of compression and centroid of tension to the depth (d).

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

t_2 = Thickness of flat slab without drop panels, or the thickness of flat slab through the drop panels where such are used.

t_3 = Thickness of flat slab (with drop panels) at points outside the drop panel.

v = Shearing unit stress.

V = Total shear.

V' = Excess of the total shear over that permitted on the concrete.

801—Shearing Unit Stress

(*a*) The shearing unit stress (v), as a measure of diagonal tension, in reinforced concrete flexural members shall be computed by formula (12):

$$v = \frac{V}{bjd} \qquad (12)$$

(*b*) For beams of I or T section, b' shall be substituted for b in formula (12).

(*c*) In concrete joist floor construction, where burned clay or concrete tile are used, b' may be taken as a width equal to the thickness of the concrete web plus the thicknesses of the vertical shells of the concrete or burned clay tile in contact with the joist as in Section 708(b).

(*d*) When the value of the shearing unit stress computed by formula (12) exceeds the shearing unit stress (v_c) permitted on the concrete of an unreinforced web (see Section 305(a)), web reinforcement shall be provided to carry the excess.

802—Types of Web Reinforcement

(*a*) Web reinforcement may consist of:

1. Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

2. Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30 degrees or more thereto.

3. Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.

4. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the concrete by the reinforcement (See Section 804(f)).

(*b*) Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Section 904.

803—Stirrups

(*a*) The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by formula (13).

$$A_v = \frac{V's}{f_v jd} \qquad (13)$$

(*b*) Inclined stirrups shall be proportioned by formula (15) (Section 804(d).

(*c*) Stirrups placed perpendicular to the longitudinal reinforcement shall not be used alone as web reinforcement when the shearing unit stress (v) exceeds $0.08f'_c$.

804—Bent Bars

(*a*) When the web reinforcement consists of a single bent bar or of a single group of bent bars the required area of such bars shall be computed by formula (14).

$$A_v = \frac{V'}{f_v \sin \alpha} \qquad (14)$$

(*b*) In formula (14) V' shall not exceed $0.040\, f'_c\, bjd$.

(*c*) Only the center three-fourths of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

(*d*) Where there is a series of parallel bent bars, the required area shall be determined by formula (15).

$$A_v = \frac{V's}{f_v jd\ (\sin \alpha + \cos \alpha)} \qquad \text{.....(15)}$$

(*e*) When bent bars, having a radius of bend of not more than two times the diameter of the bar are used alone as web reinforcement, the allowable shearing unit stress shall not exceed $0.060\ f'_c$. This shearing unit stress may be increased at the rate of $0.01\ f'_c$ for each increase of four bar diameters in the radius of bend until the maximum allowable shearing unit stress is reached. (See Section 305(a).)

(*f*) The shearing unit stress permitted when special arrangements of bars are employed shall be that determined by making comparative tests, to destruction, of specimens of the proposed system and of similar specimens reinforced in conformity with the provisions of this code, the same factor of safety being applied in both cases.

805—Combined Web Reinforcement

(*a*) Where more than one type of reinforcement is used to reinforce the same portion of the web, the total shearing resistance of this portion of the web shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once, and no one type of reinforcement shall be assumed to resist more than $\frac{2\ V'}{3}$.

806—Spacing of Web Reinforcement

(*a*) Where web reinforcement is required it shall be so spaced that every 45 degree line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement. If a shearing unit stress in excess of $0.06\ f'_c$ is used, every such line shall be crossed by at least two such lines of web reinforcement.

807—Shearing Stress in Flat Slabs

(*a*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_2 - 1\frac{1}{2}$ in. beyond the edge of the column capital

and parallel with it, shall not exceed the following values when computed by formula (12) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.):

1. $0.03\ f'_c$, when at least 50 per cent of the total negative reinforcement in the column strip passes directly over the column capital.

2. $0.025\ f'_c$, when 25 per cent or less of the total negative reinforcement in the column strip passes directly over the column capital.

3. For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(*b*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_3 - 1\frac{1}{2}$ in. beyond the edge of the drop panel and parallel with it shall not exceed $0.03\ f'_c$ when computed by formula (12) (in which d shall be taken as $t_3 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in the column strip must be within the width of strip directly above the drop panel.

808—Shear and Diagonal Tension in Footings

(*a*) In isolated footings the shearing unit stress computed by formula (12) on the critical section (see 1205(a)), shall not exceed $0.03\ f'_c$, nor in any case shall it exceed 75 p.s.i.

Chapter IX—Bond and Anchorage

900—Notation

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

j = Ratio of distance between centroid of compression and centroid of tension to the depth (d).

Σo = Sum of perimeters of bars in one set.

u = Bond stress per unit of surface area of bar.

V = Total shear.

901—Computation of Bond Stress in Beams

(*a*) In flexural members in which the tensile reinforcement is parallel to the compression face, the bond stress at any cross section shall be computed by formula (16).

$$u = \frac{V}{\Sigma o\, jd} \qquad (16)$$

in which V is the shear at that section.

(*b*) Adequate end anchorage shall be provided for the tensile reinforcement in all flexural members to which formula (16) does not apply, such as footings, brackets and other tapered or stepped beams in which the tensile reinforcement is not parallel to the compression face.

902—Ordinary Anchorage Requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar shall be extended at least twelve diameters beyond the point at which it is no longer needed to resist stress. In cases where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond, the bar shall extend into a region of compression and be anchored by means of a standard hook or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the

bar and either made continuous with the positive reinforcement or anchored in a region of compression.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of ten or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks, or other adequate anchorage.

(*c*) In simple beams, or at the outer ends of freely supported end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of twelve or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

903—Special Anchorage Requirements

(*a*) Where increased shearing or bond stresses are permitted because of the use of special anchorage (See Section 305), every bar shall be terminated in a standard hook in a region of compression, or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the negative or positive reinforcement.

904—Anchorage of Web Reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

1. Welding to longitudinal reinforcement.

2. Hooking tightly around the longitudinal reinforcement through 180 degrees.

3. Embedment above or below the mid-depth of the beam on the compression side, a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed $.04\,f'_c$ on plain bars nor $.05\,f'_c$ on deformed bars.

4. Standard hook (see Section 906(a)), considered as developing 10,000 p.s.i., plus embedment sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified in Section 305(a), Table 305(a). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

(*b*) The extreme ends of bars forming simple U or multiple stirrups shall be anchored by one of the methods of Section 904(a) or

shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least twelve diameters of the stirrup bar.

(*c*) The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees, or by being welded or otherwise rigidly attached thereto.

(*d*) Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

1. As specified in Section 904(a), (3).

2. As specified in Section 904(a), (4).

3. By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam, plus an extension of the bar of not less than twelve diameters of the bar terminating in a standard hook. This short radius bend extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

4. By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

5. The tensile unit stress at the beginning of a bend may be increased from 10,000 p.s.i. when the radius of bend is two bar diameters, at the rate of 1,000 p.s.i. tension for each increase of 1½ bar diameters in the radius of bend, provided that the length of the bar in the bend and extension is sufficient to develop this increased tensile stress by bond at the unit stresses given in Section 904(e), (3).

(*f*) In all cases web reinforcement shall be carried as close to the compression surface of the beam as fireproofing regulations and the proximity of other steel will permit.

905—Anchorage of Bars in Footing Slabs

(*a*) All bars in footing slabs shall be anchored by means of standard hooks. The outer faces of these hooks shall be not less than three inches nor more than six inches from the face of the footing.

906—Hooks

(*a*) The terms "hook" or "standard hook" as used herein shall mean either

1. A complete semicircular turn with a radius of bend on the axis of the bar of not less than three and not more than six bar diameters, plus an extension of at least four bar diameters at the free end of the bar, or

2. A 90° bend having a radius of not less than four bar diameters plus an extension of twelve bar diameters.

Hooks having a radius of bend of more than six bar diameters shall be considered merely as extensions to the bars, and shall be treated as in Section 904(e), (5).

(*b*) In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(*c*) No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 10,000 p.s.i.

(*d*) Hooks shall not be considered effective in adding to the compressive resistance of bars.

(*e*) Any mechanical device capable of developing the strength of the bar without damage to the concrete may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

Chapter X—Flat Slabs—with Square or Rectangular Panels

1000—Notation

A = The distance from the center line of the column, in the direction of any span, to the intersection of a 45-degree diagonal line from the center of the column to the bottom of the flat slab or drop panel, where such line lies wholly within the column, capital, or bracket, provided such capital or bracket is structurally capable of resisting shears and moments without excessive unit stress. In no case shall A be greater than one-eighth the span in the direction considered.

A_{av} = Average of the two values of A for the two columns at the ends of a column strip, in the direction of the spans considered.

c = Diameter or width of column capital at the under side of the slab or drop panel. No portion of the column capital shall be considered for structural purposes which lies outside the largest right circular cone, with 90 degrees vertex angle, that can be included within the outlines of the column capital.

L = Span length of slab center to center of columns in the direction of which bending is considered.

M_o = Sum of the positive and the average negative bending moments at the critical design sections of a flat slab panel. See Section 1003(b).

W = Total dead and live load uniformly distributed over a single panel area.

x = Coefficient of span (L) which gives the distance from the center of column to the critical section for negative bending in design according to Section 1002(a).

1001—Scope

(*a*) The term flat slab shall mean a reinforced concrete slab supported by columns with or without flaring heads or column capitals, with or without depressed or drop panels and generally without beams or girders.

(*b*) Recesses or pockets in flat slab ceilings, located between reinforcing bars and forming cellular or two-way ribbed ceilings, whether

left open or filled with permanent fillers, shall not prevent a slab from being considered a flat slab; but allowable unit stresses shall not be exceeded.

(*c*) This chapter provides for two methods of design of flat slab structures.

1. Any type of flat slab construction may be designed by application of the principles of continuity, using the method outlined in Section 1002, or using other recognized methods of elastic analysis. In either case, the design must be subject to the provisions of Sections 1006, 1008 and 1009.

2. The common cases of flat slab construction described in Section 1003 may be designed by the use of moment coefficients, given in Sections 1003 and 1004, and subject to the provisions of Sections 1005, 1006, 1007, 1008 and 1009.

1002—Design of Flat Slabs as Continuous Frames

(*a*) Except in the cases of flat slab construction where specified coefficients for bending may be used, as provided in Section 1003, bending and shear in flat slabs and their supports shall be determined by an analysis of the structure as a continuous frame, and all sections shall be proportioned to resist the moments and shears thus obtained. In the analysis, the following assumptions may be made:

1. The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported slabs, each strip bounded laterally by the center line of the panel on either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.

2. Each such bent may be analyzed in its entirety; or each floor thereof and the roof may be analyzed separately with its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom beyond which the slab continues.

3. The joints between columns and slabs may be considered rigid and this rigidity may be assumed to extend in the slabs a distance A from the center of the columns, and in the column to the intersection of the sides of the column and the 45 degree line defining A. The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected.

Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and shear.

4. The supporting columns may be assumed free from settlement or lateral movement unless the amount thereof can be reasonably determined.

5. The moment of inertia of slab or column at any cross-section may be assumed to be that of the gross section of the concrete. Variation in the moments of inertia of the slabs and columns along their axes shall be taken into account.

6. Where the load to be supported is definitely known, the structure shall be analyzed for that load. Where the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum bending may be assumed to obtain at all sections under full live load. Elsewhere, maximum positive bending near mid-span of a panel may be assumed to obtain under full live load in the panel and in alternate panels; and maximum negative bending at a support may be assumed to obtain under full live load in the adjacent panels only.

7. Where neither beams nor girders help to transfer the slab load to the supporting column, the critical section for negative bending may be assumed as not more than the distance xL from the column center, where

$$x = 0.073 + 0.57\frac{A}{L} \quad \ldots\ldots\ldots\

10. The maximum bending in columns may be assumed to obtain under full live load in alternate panels. Columns shall be proportioned to resist the maximum bending combined with the maximum direct load consistent therewith; and for maximum direct load combined with the bending under full load, the direct load subject to allowable reductions, in the manner provided in Chapter 11.

(*b*) The foregoing provisions outline the method to be followed in analyzing and designing flat slabs in the general case. In all instances the design must conform to the requirements for panel strips and critical design sections, slab thickness and drop panels, capitals and brackets, arrangement of reinforcement and openings in flat slabs, as provided in Sections 1006 to 1009 inclusive.

1003—Design of Flat Slabs by Moment Coefficients

(*a*) In those cases of flat slab construction which fall within the following limitations as to continuity and dimensions, the bending moments at critical sections may be determined by the use of specified coefficients as provided in Section 1004.

1. The ratio of length to width of panel does not exceed 1.33.

2. The slab is continuous for at least three panels in each direction.

3. The successive span lengths in each direction differ by not more than twenty per cent of the shorter span.

(*b*) In such slabs, the numerical sum of the positive and negative bending moments in the direction of either side of an interior rectangular panel shall be assumed as not less than

$$M_o = 0.09\, WL \left(1 - \frac{2c}{3L}\right)^2 \qquad (19)$$

(*c*) Three-fourths of the width of the strip shall be taken as the width of the section in computing compression due to bending, except that, on a section through a drop panel, three-fourths of the width of the drop panel shall be taken. Account shall be taken of any recesses which reduce the compressive area.

(*d*) The design of slabs under the procedure given in this section is subject to the provisions of all subsequent sections of this chapter (Sections 1004 to 1009).

1004—Bending Moment Coefficients

(*a*) The bending moments at the critical sections of the middle and column strips of an interior panel shall be assumed as given in Table 1004(a).

TABLE 1004(*a*)—BENDING MOMENTS IN INTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Negative moment	$0.50M_o$
	Positive moment	$0.20M_o$
Middle strip	Negative moment	$0.15M_o$
	Positive momtnt	$0.15M_o$
Without drop panel		
Column strip	Negative moment	$0.46M_o$
	Positive moment	$0.22M_o$
Middle strip	Negative moment	$0.16M_o$
	Positive moment	$0.16M_o$

TABLE 1004(*b*)—BENDING MOMENTS IN EXTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Exterior negative	$0.45M_o$
	Positive moment	$0.25M_o$
	Interior negative	$0.55M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.19M_o$
	Interior negative	$0.165M_o$
Without drop panel		
Column strip	Exterior negative	$0.41M_o$
	Positive moment	$0.28M_o$
	Interior negative	$0.50M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.20M_o$
	Interior negative	$0.176M_o$

TABLE 1004(*c*)—BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS

		Marginal Beams with Depth greater than 1½ times the Slab Thickness; or Bearing Wall.		Marginal Beams with depth 1½ times the Slab Thickness or less.	
(a) Load to be carried by Marginal Beam or Wall		Loads directly superimposed upon it plus a uniform load equal to one-quarter of the total live and dead panel load.		Loads directly superimposed upon it exclusive of any panel load.	
		With Drop	Without Drop	With Drop	Without Drop
(b) Moment to be used in the design of Half Column Strip adjacent and parallel to Marginal Beam or Wall.	Neg.	$0.125M_o$	$0.115M_o$	$0.25M_o$	$0.23M_o$
	Pos.	$0.05M_o$	$0.055M_o$	$0.10M_o$	$0.11M_o$
(c) Negative Moment to be used in Design of Middle Strip continuous across a Beam or Wall	Neg.	$0.195M_o$	$0.208M_o$	$0.15M_o$	$0.16M_o$

(*b*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the

exterior supports consist of reinforced concrete columns or reinforced concrete bearing walls integral with the slab, the ratio of stiffness of the support to that of the slab being at least as great as the ratio of the live load to the dead load and not less than one, shall be assumed as given in Table 1004(b). Where a flat slab is so supported by a wall providing restraint at the discontinuous edge, the coefficient for negative bending at the edge shall be assumed more nearly equal in the column and middle strips, the sum remaining as given in Table 1004(b), but that for the column strip shall not be less than 0.30 M_o. Bending in middle strips parallel to a discontinuous edge, except in a corner panel, shall be assumed the same as in an interior panel. M_o shall be determined as provided in Section 1003(b) for an interior panel.

(*c*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports are masonry bearing walls or other construction which provide only negligible restraint to the slab, shall be assumed as given in Table 1004(b) with the following modifications.

1. On critical sections at the face of the exterior support, negative bending in each strip shall be assumed as 0.05 M_o.

2. The coefficients for positive bending shall be increased by forty per cent.

3. The coefficients for negative bending at the first interior columns shall be increased thirty per cent.

(*d*) The bending moments in panels with marginal beams or walls, in the strips parallel and close thereto, and in the beams, shall be determined upon the basis of assumptions presented in Table 1004(c).

(*e*) For design purposes any of the moment coefficients of Tables 1004(a), 1004(b), and 1004(c) may be varied by not more than six per cent, but the numerical sum of the positive and negative moments in a panel shall not be taken as less than the amount specified.

(*f*) Panels supported by marginal beams on opposite edges shall be designed as solid one or two-way slabs to carry the entire panel load.

(*g*) The ratio of reinforcement in any strip shall not be less than 0.0025.

GENERAL REQUIREMENTS

1005—Panel Strips and Critical Design Sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one half panel in width, symmetrical about panel center line and extending through the panel in the direction of the span for bending.

A column strip consisting of the two adjacent quarter-panels either side of the column center lines.

(*b*) The critical sections for bending are located as follows:

Sections for negative bending shall be taken along the edges of the panel, on column center lines between capitals and around the perimeters of column capitals.

Sections for positive bending shall be taken at mid-span of the strips.

(*c*) Only the reinforcement which crosses a critical section within a strip may be considered effective to resist bending in the strip at that section. Reinforcement which crosses such section at an angle with the center-line of the strip shall be assumed to contribute to the resistance of bending only its effective area in the direction of the strip, as defined in Chapter 1.

1006—Slab Thickness and Drop Panels

(*a*) The thickness of a flat slab and the size and thickness of the drop panel, where used, shall be such that the compressive stress due to bending at the critical sections of any strip and the shear about the column capital and the drop panel shall not exceed the unit stresses allowed in concrete of the quality used.

(*b*) The shearing stresses in the slab outside the capital or drop panel shall be computed as provided in Section 807.

(*c*) Slab thickness shall not, however, be less than

$$\frac{L}{40} \text{ with drop panels}$$

or

$$\frac{L}{36} \text{ without drop panels}$$

(*d*) The thickness of the drop panel below the slab shall not be more than one-fourth the distance from the edge of the column capital to the edge of the drop panel.

1007—Capitals and Brackets

(*a*) Where a column is without a flaring concrete capital the distance *c* shall be taken as the diameter of the column. Structural metal embedded in the slab or drop panel may be regarded as contributing to resistance in bending and shear.

(*b*) Where a reinforced concrete beam frames into a column without capital or bracket on the same side with the beam, the value of *c* may be taken as the width of the column plus twice the projection of the beam above or below the slab or drop panel for computing bending in strips parallel to the beam.

(*c*) Brackets capable of transmitting the negative bending and the shear in the column strips to the columns without excessive unit stress may be substituted for column capitals at exterior columns. The value of *c* where brackets are used shall be taken as twice the distance from the center of the column to a point where the bracket is 1½ inches thick, but not more than the thickness of the column plus twice the depth of the bracket.

(*d*) The average of the diameters *c* of the column capitals at the four corners of a panel shall be used in determining the bending in the middle strips of the panel. The average of the diameters *c* of the two column capitals at the ends of a column strip shall be used in determining bending in the strip.

1008—Arrangement of Reinforcement

(*a*) Slab reinforcement shall be provided to resist the bending and bond stresses not only at critical sections, but also at intermediate sections.

(*b*) Bars shall be spaced evenly across strips or bands and the spacing shall not exceed three times the slab thickness.

(*c*) In exterior panels the reinforcement perpendicular to the discontinuous edge for positive bending, shall extend to the edge and have embedment of at least six inches in spandrel beams, walls or columns. All such reinforcement for negative bending shall be bent, hooked or otherwise anchored in spandrel beams, walls or columns.

1009—Openings in Flat Slabs

Openings of any size may be cut through a flat slab if provision is made for the total positive and negative resisting moments, as required in Sections 1002, 1003, without exceeding the allowable stresses as given in Sections 305 and 306.

Chapter XI—Reinforced Concrete Columns and Walls

1100—Notation

A_c = Area of core of a spirally reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

C = Ratio of allowable concrete stress, f_a, in axially loaded column to allowable fiber stress for concrete in flexure.

$D = \dfrac{l^2}{2R^2}$ = a factor, usually varying from 3 to 9. (The term R as used here is the radius of gyration of the column section.)

d = The least lateral dimension of a concrete column.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$F = \dfrac{\text{Yield point of pipe}}{45{,}000}$ (See Section 1106(b)).

f_a = Allowable unit stress in the concrete of an axially loaded reinforced concrete column.

f_c = Computed concrete fiber stress in an eccentrically loaded column.

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

f_p = Maximum allowable concrete fiber stress in an eccentrically loaded column.

f_r = Allowable unit stress in the metal core of a composite column.

f'_r = Allowable unit stress on unencased steel columns and pipe columns.

f_s = Nominal working stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

h = Unsupported length of column.

K = Least radius of gyration of a metal pipe section (in pipe columns).

$n = \frac{30{,}000}{f'_c}$

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g (See Section 1103).

P = Total allowable axial load on a column whose length does not exceed ten times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

R = Least radius of gyration of a section; ratio of gross area to core area of a spirally reinforced concrete column, A_g/A_c.

t = Overall depth of section (See Section 1109).

1101—Limiting Dimensions

(*a*) The following sections on reinforced concrete and composite columns, except Section 1107(a), apply to a short column for which the unsupported length is not greater than ten times the least dimension. When the unsupported length exceeds this value, the design shall be modified as shown in Section 1107 (a). Principal columns in buildings shall have a minimum diameter of twelve inches, or in the case of rectangular columns, a minimum thickness of ten inches, and a minimum gross area of 120 sq. in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of six inches.

1102—Unsupported Length of Columns

(*a*) For purposes of determining the limiting dimensions of columns, the unsupported length of reinforced concrete columns shall be taken as the clear distance between floor slabs, except that

1. In flat slab construction, it shall be the clear distance between the floor and the lower extremity of the capital.

2. In beam and slab construction, it shall be the clear distance between the floor and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

3. In columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane; provided that to be an adequate support, two such struts shall meet the column at approximately the same level, and the angle

between vertical planes through the struts shall not vary more than 15 degrees from a right angle. Such struts shall be of adequate dimensions and anchorage to restrain the column against lateral deflection.

4. In columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the column.

(*b*) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

1103—Spirally Reinforced Columns

(*a*) *Allowable Load*—The maximum allowable axial load, P, on columns with closely spaced spirals enclosing a circular concrete core reinforced with longitudinal bars shall be that given by Formula (20).

$$P = A_g\,(0.225\,f'_c + f_s p_g) \qquad (20)$$

Wherein A_g = the gross area of the column

f'_c = compressive strength of the concrete

f_s = nominal working stress in vertical column reinforcement, to be taken at forty per cent of the minimum specification value of the yield point; viz., 16,000 p.s.i. for intermediate grade steel and 20,000 p.s.i. for rail or hard grade steel.*

p_g = ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.

(*b*) *Vertical Reinforcement*—The ratio p_g shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be six, and the minimum diameter shall be $\frac{5}{8}$ in. The center to center spacing of bars within the periphery of the column core shall not be less than $2\frac{1}{2}$ times the diameter for round bars or three times the side dimension for square bars. The clear spacing between bars shall not be less than $1\frac{1}{2}$ inches or $1\frac{1}{2}$ times the maximum size of the coarse aggregate used. These spacing rules also apply to adjacent pairs of bars at a lapped splice; each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars.

(*c*) *Splices in Vertical Reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

*Nominal working stresses for reinforcement of higher yield point may be established at forty per cent of the yield point stress, but not more than 30,000 p.s.i., when the properties of such reinforcing steels have been definitely specified by standards of A.S.T.M. designation. If this is done, the lengths of splice required by Section 1103(c) shall be increased accordingly.

1. For deformed bars—with concrete having a strength of 3000 p.s.i. or above, twenty-four diameters of bar of intermediate grade steel and thirty diameters of bar of hard grade steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the nominal working stress. When the concrete strengths are less than 3000 p.s.i., the amount of lap shall be one-third greater than the values given above.

2. For plain bars—the minimum amount of lap shall be twenty-five per cent greater than that specified for deformed bars.

3. Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds 1¼ in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

4. Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded by a concrete capital, floor slab or by metal ties or reinforcing spirals. Where bars are offset, the slope of the inclined portion from the axis of the column shall not exceed 1 in 6 and the bars above and below the offset shall be parallel to the axis of the column.

(*d*) *Spiral Reinforcement*—The ratio of spiral reinforcement, p', shall not be less than the value given by Formula (21).

$$p' = 0.45\ (R - 1)\ \frac{f'_c}{f'_s} \qquad (21)$$

Wherein p' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

R = ratio of gross area to core area of column, A_g/A_c.

f'_s = useful limit stress of spiral reinforcement, to be taken as 40,000 p.s.i. for hot rolled rods of intermediate grade, 50,000 p.s.i. for rods of hard grade, and 60,000 p.s.i. for cold drawn wire.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spirals shall be of such size and so assembled as to permit handling and placing without being distorted from the designed dimensions. The material used in spirals shall have a minimum diameter of ¼ in. for rolled bars or No. 4 W. & M. gage for drawn wire. Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral rod or wire at each end of the spiral unit. Splices, when

necessary shall be made in spiral rod or wire by welding or by a lap of 1½ turns. The center to center spacing of the spirals shall not exceed one-sixth of the core diameter. The clear spacing between spirals shall not exceed three in. nor be less than 1⅜ in. or 1½ times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the floor level in any story or from the top of the footing in the basement, to the level of the lowest horizontal reinforcement in the slab, drop panel or beam above. In a column with a capital, it shall extend to a plane at which the diameter or width of the capital is twice that of the column.

(*e*) *Protection of Reinforcement*—The column reinforcement shall be protected everywhere by a covering of concrete cast monolithically with the core, for which the thickness shall not be less than 1½ in. nor less than 1½ times the maximum size of the coarse aggregate, nor shall it be less than required by the fire protection and weathering provisions of Section 507.

(*f*) *Isolated Column with Multiple Spirals*—In case two or more interlocking spirals are used in a column, the outer boundary of the column shall be taken as a rectangle of which the sides are outside the extreme limits of the spiral at a distance equal to the requirements for protective covering, as listed in Section 507.

(*g*) *Limits of Section of Column Built Monolithically with Wall*—For a spiral column built monolithically with a concrete wall or pier, the outer boundary of the column section shall be taken either as a circle at least 1½ in. outside the column spiral or as a square or rectangle of which the sides are at least 1½ in. outside the spiral or spirals.

(*h*) *Equivalent Circular Columns*—As an exception to the general procedure of utilizing the full gross area of the column section, it shall be permissible to design a circular column and to build it with a square, octagonal, or other shaped section of the same least lateral dimension. In such case, the allowable load, the gross area considered, and the required percentages of reinforcement shall be taken as those of the circular column.

1104—Tied Columns

(*a*) *Allowable Load*—The maximum allowable axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80 per cent of that given by Formula (20). The ratio, p_g, to be considered in tied columns shall not be less than 0.01 nor more than 0.04. The longitudinal reinforcement shall consist of at least four bars, of minimum diameter of ⅝ inch. Splices in reinforcing bars shall be made as described in Section 1103 (c).

(*b*) *Lateral Ties*—Lateral ties shall be at least ¼ in. in diameter and shall be spaced apart not over 16 bar diameters, 48 tie diameters or the least dimension of the column. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar is held firmly in its designed position and has lateral support equivalent to that provided by a 90-degree corner of a tie.

(*c*) *Limits of Column Section*—In a tied column which for architectural reasons has a larger cross section than required by considerations of loading, a reduced effective area, A_g, not less than one-half of the total area may be used in applying the provisions of Section 1104 (a).

1105—Composite Columns

(*a*) *Allowable Load*—The allowable load on a composite column, consisting of a structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by Formula (22).

$$P = 0.225\,A_c f'_c + f_s A_s + f_r A_r \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Wherein A_c = net area of concrete section
$= A_g - A_s - A_r$
A_s = cross-sectional area of longitudinal bar reinforcement.
A_r = cross-sectional area of the steel or cast-iron core.
f_r = allowable unit stress in metal core, not to exceed 16,000 p.s.i. for a steel core; or 10,000 p.s.i. for a cast-iron core.

The remaining notation is that of Section 1103.

(*b*) *Details of Metal Core and Reinforcement*—The cross-sectional area of the metal core shall not exceed 20 per cent of the gross area of the column. If a hollow metal core is used it shall be filled with concrete. The amounts of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shell outside the spiral shall conform to the limiting values specified in Sections 1103 (b), (c) and (d). A clearance of at least three inches shall be maintained between the spiral and the metal core at all points except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to two inches.

(*c*) *Splices and Connections of Metal Cores*—Metal cores in composite columns shall be accurately milled at splices and positive provision shall be made for alignment of one core above another. At the column base, provision shall be made to transfer the load to the footing at safe unit stresses in accordance with Section 305 (a). The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal

as to leave ample section of concrete above the base for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members such as billets, brackets or other positive connections; these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of Formula (22) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, in accordance with Formula (20), all floor loads brought onto the column at levels between the metal brackets or connections. In applying Formula (20), the value of A_g shall be interpreted as the area of the concrete section outside the metal core, and the allowable load on the reinforced concrete section shall be further limited to $0.35\ f'_c A_g$. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders.

(*d*) *Allowable Load on Metal Core Only*—The metal cores of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

1106—Combination Columns

(*a*) *Steel Columns Encased in Concrete*—The allowable load on a structural steel column which is encased in concrete at least 2½ inches thick over all metal (except rivet heads) reinforced as hereinafter specified, shall be computed by Formula (23).

$$P = A_r f'_r \left[1 + \frac{A_g}{100\ A_r}\right] \qquad (23)$$

Wherein A_r = cross-sectional area of steel column.
f'_r = allowable stress for unencased steel column.
A_g = total area of concrete section.

The concrete used shall develop a compressive strength, f'_c, of at least 2000 p.s.i. at 28 days. The concrete shall be reinforced by (the equivalent of) welded wire mesh having wires of No. 10W. and M. gage, the wires encircling the column being spaced not more than four inches apart and those parallel to the column axis not more than eight inches apart. This mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface and shall be lap-spliced at least forty wire diameters and wired at the splice. Special brackets shall be used to receive the entire floor load at each floor level. The steel column shall be designed to carry safely any construction or other loads to be placed upon it prior to its encasement in concrete.

(*b*) *Pipe Columns*—The allowable load on columns consisting of steel pipe filled with concrete shall be determined by Formula (24).

$$P = 0.225 f'_c A_c + f'_r A_r \quad \ldots\ldots (24)$$

The value of f'_r shall be given by Formula (25).

$$f'_r = \left[18{,}000 - 70 \frac{h}{K}\right] F \quad \ldots\ldots (25)$$

Wherein f'_r = allowable unit stress in metal core
h = unsupported length of column
K = least radius of gyration of metal core section

$$F = \frac{\text{yield point of pipe}}{45{,}000}$$

If the yield point of the pipe is not known, the factor F shall be taken as 0.5.

1107—Long Columns

(*a*) The maximum allowable load P' on axially loaded reinforced concrete or composite columns having a length, h, greater than ten times the least lateral dimension, d, shall be given by Formula (26).

$$P' = P\left[1.3 - .03 \frac{h}{d}\right] \quad \ldots\ldots (26)$$

where P is the allowable axial load on a short column as given by Formulas (20) and (22).

The maximum allowable load P' on eccentrically loaded columns in which $\frac{h}{d}$ exceeds ten shall also be given by Formula (26), in which P is the allowable eccentrically applied load on a short column as determined by the provisions of Sections 1109 and 1110. In long columns subjected to definite bending stresses, as determined in Section 1108, the ratio $\frac{h}{d}$ shall not exceed twenty.

1108—Bending Moments in Columns

(*a*) The bending moments in the columns of all reinforced concrete structures shall be determined on the basis of loading conditions and restraint and shall be provided for in the design. When the stiffness and strength of the columns are utilized to reduce moments in beams, girders, or slabs, as in the case of rigid frames, or in other forms of continuous construction wherein column moments are unavoidable, they shall be provided for in the design. In building frames, particular

attention shall be given to the effect of unbalanced floor loads on both exterior and interior columns and of eccentric loading due to other causes. Wall columns shall be designed to resist moments produced by

1. Loads on all floors of the building
2. Loads on a single exterior bay at two adjacent floor levels, or
3. Loads on a single exterior bay at one floor level

Resistance to bending moments at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffnesses and conditions of restraint.

1109—Determination of Combined Axial and Bending Stresses

(*a*) In a reinforced concrete column, designed by the methods of this Chapter, which is (1) symmetrical about two perpendicular planes through its axis and (2) subject to an axial load, N, combined with bending in one or both of the planes of symmetry (but with the ratio of eccentricity to depth, e/t, no greater than 1.0 in either plane), the combined fiber stress in compression may be computed on the basis of recognized theory applying to uncracked sections, using Formula 27.

$$f_c = \frac{N}{A_g}\left[\frac{1 + \frac{De}{t}}{1 + (n-1)p_g}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Equating this calculated stress, f_c, to the allowable stress, f_p, in Formula 29, it follows that the column can be designed for an equivalent axial load, P, as given by Formula 28.*

$$P = N\left[1 + \frac{CDe}{t}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

When bending exists on both axes of symmetry, the quantity $\frac{De}{t}$ is to be computed as the numerical sum of the $\frac{De}{t}$ quantities in the two directions.

(*b*) For columns in which the load, N, has an eccentricity, e, greater than the column depth, t, or for beams subject to small axial loads, the determination of the fiber stress f_c shall be made by use of recognized theory for cracked sections, based on the assumption that no tension exists in the concrete. For such cases the tensile steel stress shall also be investigated.

*For approximate or trial computations, D may be taken as eight for a circular spiral column and as five for a rectangular tied or spiral column.

1110—Allowable Combined Axial and Bending Stress

(*a*) For spiral and tied columns, eccentrically loaded or otherwise subjected to combined axial compression and flexural stress, the maximum allowable compressive stress, f_p, is given by Formula (29).

$$f_p = f_a \left[\frac{1 + \frac{De}{t}}{1 + \frac{CDe}{t}} \right] = f_a \left[\frac{t + De}{t + CDe} \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Wherein the notation is that of Section 1103 and 1109, and, in addition f_a is the average allowable stress on an axially loaded reinforced concrete column, and C is the ratio of f_a to the allowable fiber stress for members in flexure. Thus $f_a = \frac{0.225\, f'_c + f_s p_g}{1 + (n - 1)\, p}$ for spiral columns and 0.8 of this value for tied columns, and $C = \frac{f_a}{0.45\, f'_c}$.

1111—Wind Stresses

(*a*) When the allowable stress in columns is modified to provide for combined axial load and bending, and the stress due to wind loads is also added, the total shall still come within the allowable values specified for wind loads in Section 603 (c).

1112—Monolithic Walls

(*a*) The allowable working stresses in reinforced concrete bearing walls with minimum reinforcement as required by Section 1112(i), shall be $0.25f'_c$ for walls having a ratio of height to thickness of ten or less, and shall be reduced proportionally to $0.15f'_c$ for walls having a ratio of height to thickness of twenty-five. When the reinforcement in bearing walls is designed, placed and anchored in position as for tied columns, the allowable working stresses shall be on the basis of Section 1104, as for columns. In the case of concentrated loads, the length of the wall to be considered as effective for each shall not exceed the center to center distance between loads, nor shall it exceed the width of the bearing plus four times the wall thickness. The ratio p_g shall not exceed 0.04.

(*b*) Walls shall be designed for any lateral or other pressure to which they are subjected. Proper provision shall be made for eccentric loads and wind stresses. In such designs the allowable stresses shall be as given in Section 305(a) and 603(c).

(*c*) Panel and enclosure walls of reinforced concrete shall have a thickness of not less than five inches and not less than one thirtieth the distance between the supporting or enclosing members.

(*d*) Bearing walls of reinforced concrete in building of fire-resistive construction shall be not less than six inches in thickness for the uppermost fifteen feet of their height; and for each successive twenty-five feet downward, or fraction thereof, the minimum thickness shall be increased one inch. In two story dwellings the walls may be six inches in thickness throughout.

(*e*) In buildings of non-fire resistive construction bearing walls of reinforced concrete shall not be less than one and one-third times the thickness required for buildings of fire-resistive construction, except that for dwellings of two stories or less in height the thickness of walls may be the same as specified for buildings of fire-resistive construction.

(*f*) Exterior basement walls, foundation walls, fire walls and party walls shall not be less than eight inches thick whether reinforced or not.

(*g*) Reinforced concrete bearing walls shall have a thickness of at least one twenty-fifth of the unsupported height or width, whichever is the shorter; provided however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of increased thickness.

(*h*) Monolithic walls shall be anchored to the floors, columns, pilasters, buttresses and intersecting walls with reinforcement at least equivalent to three-eighths inch round bars twelve inches on centers, for each layer of wall reinforcement.

(*i*) Monolithic walls shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall, if of bars, and 0.0018 times the area if of electrically welded wire fabric.* The wire of the welded fabric shall be of not less than No. 10 W. & M. gage. Walls more than ten inches in thickness shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than two inches nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement, shall be placed not less than three-fourths inches and not more than one-third the thickness of the wall from the interior surface. Bars, if used, shall not be less than the equivalent of three-eighths inch round

*Expanded metal has been omitted until a specification can be formulated.

bars, nor shall they be spaced more than eighteen inches on centers. Welded wire[3] reinforcement for walls shall be in flat sheet form.

(*j*) In addition to the minimum as prescribed in 1112(i) there shall be not less than two five-eighths inch diameter bars around all window or door openings. Such bars shall extend at least twenty-four inches beyond the corner of the openings.

(*k*) Where reinforced concrete bearing walls consist of studs or ribs tied together by reinforced concrete members at each floor level, the studs may be considered as columns, but the restrictions as to minimum diameter or thickness of columns shall not apply.

Chapter XII—Footings

1201—Scope

(*a*) The requirements prescribed in Sections 1202 to 1209 apply only to isolated footings.*

1202—Loads and Reactions

(*a*) Footings shall be proportioned to sustain the applied loads and induced reactions without exceeding the allowable stresses as prescribed in Sections 305 and 306, and as further provided in Sections 1205, 1206 and 1207.

(*b*) In cases where the footing is concentrically loaded and the member being supported does not transmit any moment to the footing, computations for moments and shears shall be based on an upward reaction assumed to be uniformly distributed per unit area or per pile and a downward applied load assumed to be uniformly distributed over the area of the footing covered by the column, pedestal, wall, or metallic column base.

(*c*) In cases where the footing is eccentrically loaded and/or the member being supported transmits a moment to the footing, proper allowance shall be made for any variation that may exist in the intensities of reaction and applied load consistent with the magnitude of the applied load and the amount of its actual or virtual eccentricity.

(*d*) In the case of footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the center of the pile.

1203—Sloped or Stepped Footings

(*a*) In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that the allowable stresses are not exceeded at any section.

(*b*) In sloped or stepped footings, the effective cross-section in compression shall be limited by the area above the neutral plane.

(*c*) Sloped or stepped footings shall be cast as a unit.

1204—Bending Moment

(*a*) The external moment on any section shall be determined by passing through the section a vertical plane which extends completely

*The committee is not prepared at this time to make recommendations for combined footings—those supporting more than one column or wall.

across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of said plane.

(*b*) The greatest bending moment to be used in the design of an isolated footing shall be the moment computed in the manner prescribed in Section 1204 (a) at sections located as follows:

1. At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.

2. Halfway between the middle and the edge of the wall, for footings under masonry walls.

3. Halfway between the face of the column or pedestal and the edge of the metallic base, for footings under metallic bases.

(*c*) The width resisting compression at any section shall be assumed as the entire width of the top of the footing at the section under consideration.

(*d*) In one-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to the moment computed in the manner prescribed in Section 1204(a); and the reinforcement thus determined shall be distributed uniformly across the full width of the section.

(*e*) In two-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to eighty-five per cent of the moment computed in the manner prescribed in Section 1204(a); and the total reinforcement thus determined shall be distributed across the corresponding resisting section in the manner prescribed for square footings in Section 1204(f), and for rectangular footings in Sec. 1204(g).

(*f*) In two-way square footings, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

(*g*) In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. In the case of the reinforcement in the short direction, that portion determined by formula (30) shall be uniformly distributed across a band-width (*B*) centered with respect to the center line of the column or pedestal and having a width equal to the length of the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

$$\frac{\text{Reinforcement in band-width } (B)}{\text{Total reinforcement in short direction}} = \frac{2}{(S+1)} \qquad (30)$$

In formula (30), "*S*" is the ratio of the long side to the short side of the footing.

1205—Shear and Bond

(*a*) The critical section for shear to be used as a measure of diagonal tension shall be assumed as a vertical section obtained by passing a series of vertical planes through the footing, each of which is parallel to a corresponding face of the column, pedestal, or wall and located a distance therefrom equal to the depth (*d*) for footings on soil, and one-half the depth *d* for footings on piles.

(*b*) Each face of the critical section as defined in Section 1205(a) shall be considered as resisting an external shear equal to the load on an area bounded by said face of the critical section for shear, two diagonal lines drawn from the column or pedestal corners and making 45° angles with the principal axes of the footing, and that portion of the corresponding edge or edges of the footing intercepted between the two diagonals.

(*c*) Critical sections for bond shall be assumed at the same planes as those prescribed for bending moment in Section 1204(b); also at all other vertical planes where changes of section or of reinforcement occur.

(*d*) Computations for shear to be used as a measure of bond shall be based on the same section and loading as prescribed for bending moment in Section 1204(a).

(*e*) The total tensile reinforcement at any section shall provide a bond resistance at least equal to the bond requirement as computed from the following percentages of the external shear at the section:

1. In one-way reinforced footings, 100 per cent.
2. In two-way reinforced footings, 85 per cent.

(*f*) In computing the external shear on any section through a footing supported on piles, the entire reaction from any pile whose center is located six inches or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose center is located six inches or more inside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight-line interpolation between full value at six inches outside the section and zero value at six inches inside the section.

(*g*) For allowable shearing stresses, see Section 305 and 808.

(*h*) For allowable bond stresses, see Section 305 and 901 to 905.

1206—Transfer of Stress at Base of Column

(*a*) The stress in the longitudinal reinforcement of a column or pedestal shall be transferred to its supporting pedestal or footing

either by extending the longitudinal bars into the supporting member, or by dowels.

(*b*) In case the transfer of stress in the reinforcement is accomplished by extension of the longitudinal bars, they shall extend into the supporting member the distance required to transfer to the concrete, by allowable bond stress, their full working value.

(*c*) In cases where dowels are used, their total sectional area shall be not less than the sectional area of the longitudinal reinforcement in the member from which the stress is being transferred. In no case shall the number of dowels per member be less than four and the diameter of the dowels shall not exceed the diameter of the column bars by more than one-eighth inch.

(*d*) Dowels shall extend up into the column or pedestal a distance at least equal to that required for lap of longitudinal column bars (see Section 1103) and down into the supporting pedestal or footing the distance required to transfer to the concrete, by allowable bond stress, the full working value of the dowel.

(*e*) The compressive stress in the concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The unit compressive stress on the loaded area shall not exceed the bearing stress allowable for the quality of concrete in the supporting member as limited by the ratio of the loaded area to the supporting area.

(*f*) For allowable bearing stresses see Table 305(a), Section 305.

(*g*) In sloped or stepped footings, the supporting area for bearing may be taken as the top horizontal surface of the footing, or assumed as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the area actually loaded, and having side slopes of one vertical to two horizontal.

1207—Pedestals and Footings (Plain Concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on sections as prescribed in Section 1204 and 1205 for reinforced concrete footings.

1208—Footings Supporting Round Columns

(*a*) In computing the stresses in footings which support a round or octagonal concrete column or pedestal, the "face" of the column or pedestal shall be taken as the side of a square having an area equal to the area enclosed within the perimeter of the column or pedestal.

1209—Minimum Edge-Thickness

(*a*) In reinforced concrete footings, the thickness above the reinforcement at the edge shall be not less than six in. for footings on soil, nor less than twelve in. for footings on piles.

(*b*) In plain concrete footings, the thickness at the edge shall be not less than eight in. for footings on soil, nor less than fourteen in. above the tops of the piles for footings on piles.

Discussion of this report (including Convention Discussion, Washington, Feb. 18-20, 1941) should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL for June 1941.

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INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN JUNE, 1941

Discussion of the report of A. C. I. Committee 318, Standard Building Code:

Proposed Building Regulations for Reinforced Concrete*

AT THE CONVENTION AND BY J. K. PEEBLES, JR. AND THE COMMITTEE

The convention discussion, the discussion in writing (by mail) and the record of the action of the convention in adopting the proposed regulations as an A. C. I. Standard (subject to letter ballot now in process to be canvassed July 21) are here prefaced by the remarks by R. D. Bradbury, member of Committee 318, in presenting the report to the convention.

CONVENTION DISCUSSION

R. D. Bradbury: In presenting the report of this Committee, it might be well to sketch briefly first the historical background of what the Institute has been attempting to do through Committee 318 (formerly Committee 501). In the early part of 1929, if I remember correctly, or soon after adoption of the previous Standard Building Code, the Institute organized the present committee with instructions to undertake a thorough review and re-study of the subject of a suitable Building Code for the Institute. As many of you recall, the major effort of the Committee during the years immediately following its organization was devoted to a study of the Institute's elaborate and comprehensive Reinforced Concrete Column Investigation under the guidance of a special Institute Committee organized for this work. During that period, which extended over a number of years, the Standard Building Code Committee made no attempt to present a complete report.

During that interim, in addition to studying columns, the Code Committee was also busily engaged in preparing suitable require-

*JOURNAL, Amer. Concrete Inst., Nov. 1940; *Proceedings* V. 37, p. 77; presented at 37th Annual Convention on a motion for adoption as an A. C. I. Standard.

ments for other sections of the report. In due time the Committee developed a complete Code which contained many vital changes as compared to the former Code. That report was finally presented and tentatively adopted in 1936. The present Tentative Standard is the result of the work of the Committee up to that time. The ink was hardly dry on the 1936 Tentative Standard before we were all called together again and immediately undertook its complete revision from beginning to end. Each chapter was thoroughly reviewed. Considerable new matter came in then and it was through that secondary period of review that the subject of Flat Slabs was taken up and revised in a manner which, in the opinion of some, is rather radical. The next report was presented and discussed last year at the 36th Annual Convention in Chicago. There were some differences of opinion, and comments which indicated possibilities for improvement. As a result, last year's report, instead of being acted upon by the Institute, was referred back to the Committee. I think it was referred back to the Committee not so much because of any vital disagreement or dissatisfaction with the Code, as primarily for the reason that it had not been published prior to the Convention in a manner and in sufficient time to permit the members to form opinions or to register their comments. In view of the experience of last year, the Committee has again thoroughly reviewed each chapter of the proposed Code and some further revisions have been made although few are of a vital character. It is this latest revised report that is being presented today for adoption or rejection by the Institute. In submitting previous reports, it has been customary for the Committee to present them for discussion chapter by chapter. However, it seems unnecessary to follow that tedious procedure in submitting the present report, since the complete report has been published in the JOURNAL well in advance of this meeting and ample opportunity has been given for members to become familiar with its contents. Therefore, I will merely touch upon its high spots and call special attention to some of the more recent changes. Numerous editorial changes have been made, the most important of which is the placing of all notations applying to any particular chapter at the head of that chapter instead of in Chapter I. In Chapter II, Materials and Tests, no important changes have been made. In Chapter III, Concrete Quality and Working Stresses, certain revisions have been made in the allowable unit stresses in concrete. In the case of bond stresses, the same percentage values have been retained but certain top limits or ceilings have been placed on allowable bond stresses regardless of the f'_c value of the concrete. A similar limitation has also been

prescribed for the case of shear in footings when used as a measure of diagonal tension. Another change somewhat radical in comparison with current design practice as it has existed in the past and as heretofore recommended by the Institute, is a change from $0.40f'_c$ to $0.45f'_c$ as the allowable extreme fiber stress in compression under flexure. Incidentally, this change is in conformity with the latest Joint Committee recommendations.

In Chapter VII, Flexural Computations, some changes have been made in the arbitrary moment coefficients. The numeral values have been revised; also the cases to which the arbitrary coefficients are applicable have been restricted by limitation upon the difference in length of adjoining spans and the ratio of live to dead load. In the case of floors supported on four sides, the Committee has retained the same basic procedure embodied in the present tentative standard adopted in 1936. When this method of handling slabs supported on four sides was first presented some years ago, it was criticised by some who thought that it was unnecessarily elaborate and mathematically cumbersome in its application to the usual problems of design. That it is an elaborate treatment is admitted, but it has been retained in the present report because the Committee considered it advisable to prescribe a method that would provide for all possible contingencies, by making provision for unequal adjacent span lengths in either or both directions, unequal loading on adjacent spans in either or both directions, and varying conditions of continuity. As to its being mathematically cumbersome, this criticism has been amply met by the preparation of certain tables of constants and other explanatory matter presented as supplementary footnotes.

Flab Slabs, as you all know, have been given a very different treatment from that formerly used, in that provision is made for design either by prescribed moment coefficients or by elastic analysis. For certain conditions and within certain limitations, substantially the former methods may be used. In other cases, or in any case if the designer so prefers, he may resort to a form of elastic analysis which the committee suggests by prescribing certain permissible assumptions.

As to reinforced concrete columns, that subject certainly has been fed to the Institute piecemeal and in sufficient doses for the members to be thoroughly familiar with what has been done. After the 1936 Tentative Code was adopted and some hasty changes made at the Convention, the Committee thoroughly reviewed the column section and made slight revisions in the factor of safety applied to the $f'c$ term in certain formulas. Also some of the formulas applying to bending in columns were revised and amplified. Since practically

all of these changes were reported last year, the present report contains very little if anything new on the subject of columns.

In regard to monolithic walls, this section has been re-worded and somewhat broadened in scope but, insofar as fundamental requirements are concerned, no important changes have been made.

As compared with the present Tentative Code adopted in 1936, the Chapter on Footings has been completely re-written and contains a number of major revisions. These were all fully reported at the Chicago Convention last year. Among these revisions, probably the outstanding change is the abandonment of the old so-called trapezoidal method of making the bending moment calculation in footings. The present report requires the bending moment to be taken as the full statical moment on a complete section across the footing in line with certain prescribed critical sections. This and other revisions in the section on footings were fully explained and discussed at the Chicago Convention. Since then, a few subsequent revisions have been made but these are all of a minor character. I believe that covers the high spots of the report, but, if there are any questions, I shall be glad to give any further detailed information that may be requested.

In conclusion: At the time the final letter ballot was taken on this particular report, the Committee consisted of 27 members, from whom 24 votes were received, and, of the 24 voting, 23 voted in the affirmative. Considering the size of the Committee's membership, and the scope and character of its assignment, this would indicate that, in submitting its final report, the Committee is in remarkable agreement.

Mr. Chairman, on behalf of Committee 318, I submit this report with the Committee's recommendation that it be adopted as an Institute Standard, and, in order formally to open the matter for discussion, I move its adoption. (The motion was seconded.)

P. L. Rogers:* I would like to make a comment with reference to this proposed Code, on an item among its definitions. While definitions are a very minor part of the work, they are for the guidance of those concerned. I wish to call attention to the definition of concrete. The Institute is vitally concerned with concrete. We all know something about concrete, but admittedly there is much about concrete that we do not know. When we present a definition, it appears that definition of necessity should be rather general, taking into consideration the things we do know and the things we do not know. Several helpful papers have been presented at this convention showing that cementitious material other than portland cement may be used to advantage in making concrete. The government as well as private

*Riverton Lime Stone Co., Riverton, Va.

agents are using cementitious material other than portland cement in the making of concrete. Mr. Affleck in his comments last evening (at the dinner), quoted from the Institute's charter in reference to cement and cementitious material.

The definition to which I refer is on page 80. It says: "Concrete, a mixture of portland cement, fine aggregate, coarse aggregate and water." To my mind that allows only one cementitious material. Our recent experience shows I think that that is a restricted definition. I would like to suggest that it be changed to something acceptable and, I believe, without necessitating several years more of arduous work and delaying the Code which is acknowledged to be very necessary today.

Chairman Richart: We have one motion before us, of course. Do you offer that as an amendment?

Mr. Rogers: Yes sir. (It was seconded by J. A. Robertson.)

Chairman Richart: Probably then from now on our discussion should be on the amendment rather than the whole report. Do you have anything to say, Mr. Bradbury?

Mr. Bradbury: This matter recently came to the Committee as a comment, I assume from Mr. Rogers. It has been discussed informally among a few of us, Mr. Stephens and some others. I believe the feeling of the Committee is that the object of this Code is to present a set of regulations for reinforced concrete involving the use of portland cement. However, the code contains a provision under which special materials or methods may be used according to their particular properties or merit by making application through proper procedure to the building authority under which this Code would become a part of the legal requirement. If the fundamental definition of concrete is open to all types of cementitious materials, the whole code would have to be reviewed and, undoubtedly, revised in many respects to make provision for suitable allowable unit stresses, methods of design, details of construction and various other requirements that might be appropriate for a material of different properties than portland concrete.

Nolan D. Mitchell:* The gentleman I think should have read the first clause in the definitions "(a) The following terms are defined for use in this code" and following that comes the definition the gentleman read. As Mr. Bradbury says the provisions in this code are strictly for the material described as concrete by this definition.

Chairman Richart: Is there any one else who would like to discuss the amendment?

*National Bureau of Standards, Washington, D. C.

Mr. Rogers: If there is no other discussion, may I make a closing remark, that it is true, under the definition, for use in connection with this Code alone as referred to by Mr. Mitchell, and by Mr. Bradbury, I believe it would be very well in keeping with the Code for a definition more general but not too general—if we might use the definition presented by the Portland Cement Association in a publication of 1927, on design and control of concrete mixtures: "Concrete is a mixture of cement, water and inert materials put in place in a plastic condition but hardening soon after due to the process known as the hydration of the cement." I can see no objection to the use of that kind of definition.

Mr. Bradbury: I would hesitate to pass judgment hurriedly on a suggestion of that kind. It is my feeling that this Code has been drawn up in all of its phases with reference to portland cement concrete. In view of the innumerable related features of this code, I would hesitate to change blandly a fundamental definition, simply because of the effect it might possibly have on something else. I think that it is rather rash to provide hastily for some special material by definition only in the hope that it will fit in and be consistent with everything else.

John A. Robertson:* If a statement something along the line Mr. Bradbury has made is already incorporated in the Code, it would seem to solve the problem for both sides. If admixtures will not change the strength or other properties prescribed by the Code, the Code permits their use.

Mr. Rogers: There is a provision for admixtures; there may be cementitious materials, not used as admixtures but as additions.

R. W. Crum†: As I understood the motion, it was to the effect that the Committee should give consideration to using this definition. If the amendment is adopted, what are we going to do with the report as a whole? Would the report have to go back to the Committee to consider again a year from now?

Mr. Bradbury: This report is being presented today for adoption or rejection. Amendments to the report may be made at this meeting, but only on the basis of a nine-tenths affirmative vote. Now this, if it constitutes a change in substance—anything other than an apparent editorial change, would fall in the category of an amendment to the Code. It is being proposed, as I understand it, to amend the Code as submitted and if that is the case, any such changes must be decided on the basis of a nine-tenths majority vote. If it is carried, then the

*National Lime Association, Washington, D. C.
†Director, Highway Research Board, Washington, D. C.

Committee is automatically instructed to make that amendment; if it is not carried, then it is not so instructed. If, however, as the discussion progresses, there seem to be enough of these things to indicate the possibility of a majority opinion within this meeting that the whole report should be referred to the Committee for further consideration, then that procedure would require an entirely different vote.

Mr. Rogers: I propose as an amendment the adoption of the definition I previously quoted: "Concrete is a mixture of cement, water and inert materials put in place in a plastic condition but hardening soon after, through the process known as the hydration of the cement."

Chairman Richart: That is proposed then as a substitution, changing the present wording, which is as follows: "Concrete—a mixture of portland cement, fine aggregate, coarse aggregate and water."

Mr. Rogers: Yes sir.

Chairman Richart: Are there any other remarks?

Mr. Crum: I understand that that requires a nine-tenths vote.

Chairman Richart: That is correct. Are you ready for the question as stated? It will require a nine-tenths vote of members voting to adopt the amendment.

(The motion on the amendment was then put and received five affirmative votes.)

Chairman Richart: There has not been a nine-tenths vote in favor of the amendment and it is therefore lost. You still would have the right to ask this Committee to consider this question in its future deliberations.

Chairman Richart presented discussion and comment in the absence of the authors by C. A. Willson, a member of the Committee and by Arthur R. Lord.

C. A. Willson:* I hope this proposed code as published in the November 1940 JOURNAL will be approved at this convention and published as soon thereafter as possible. As you know, I have been one of the more outspoken critics of certain portions of the code, but I doubt if anything worth while will be accomplished by delaying action for another year or five years. It seems unreasonable to expect that there would ever be 100 per cent agreement among the members of such a committee. Those of us who disagree with certain portions can revise such portions to suit our own ideas and adopt the balance.

Arthur R. Lord†: When the report of this committee first appeared in the November JOURNAL, I initiated enquiries as to certain design

*Structural Engineer, Madison, Wisc., member Committee 318.
†Engineer, Chicago.

provisions dealing with flat slab construction and a correspondence has followed with several members of the committee. I regret to report that this correspondence has served only to increase my initial feeling that the Committee has incorporated into its regulations a novel design method, used only in a very few instances by one or two engineers, and not recognized in any existing building ordinance. It is based on a paper by Professor Dean Peabody presented to the Boston Society of Civil Engineers in July, 1939. While I have the highest regard for that gentleman and the highest respect for his ability, I do not accept such publication as the proper basis for A. C. I. standard building regulations, in the absence of any extended use of the method in actual design.

The introduction of this novel method results in the simultaneous recommendation by the Committee of *two* standards of design, varying by as much as 12 per cent in design strength, *for the entire field of flat slab design that has previously been covered by code regulations.* The only possible advantage of introducing a double standard of flat-slab design in the proposed standard code must lie in the fact that the new method can be applied to flat slab panels *of such extreme proportions as to have been prohibited* under previous regulations. I submit that a *new and largely untried* method should not be released in this exceedingly tricky field of design. The results may be disastrous, with the Institute standing all alone in its recommendation, as the sole sponsor.

The proposal has the further objection that it is not clearly stated, that the meaning of parts of the language used had to be explained to me, and that *economy* rather than *safety* is assumed to limit the otherwise unlimited range of the designer. As I understand it, bad designs, dangerous designs, will be expensive and uneconomical. But that will not prevent such designs from being made and built.

For the above reasons I recommend that the sections on flat slab design be referred back to the Committee, in order that they may be further seasoned before being presented to the public with Institute backing. Even in these times of uncertainty and confusion, such an adventure as this should not be incorporated in an Institute standard.

I regret that my duties with the Navy Department will not permit me to present this matter in person and to discuss it more fully as I should like to do. But I feel that I should not remain silent, even though unable to be present, when the reputation and usefulness of the Institute may be at stake.

Chairman Richart: Discussion from the floor is now in order.

M. O. Withey†: I would like to ask what the ceiling is on bond stress?

Mr. Bradbury: Those values are shown in Table 305(a) as printed on page 88 of the November JOURNAL. I wish to say, Prof. Withey, that your remarks in this connection made at the Chicago meeting were very pertinent and the Committee acted immediately upon the suggestion you made. To answer your question specifically, the ceiling is 160 p.s.i. for plain bars and 200 for deformed bars.

Professor Withey: There is no ceiling for any strength of concrete?

Mr. Bradbury: The prescribed percentage of f'_c can be used until the allowable ceiling is reached.

Professor Withey: I am not speaking now of bond; no ceiling for fiber stress and concrete in compression; 5000 lb. concrete you would permit to have a strength of 2250 p.s.i.?

Mr. Bradbury: Yes, 0.45 times its f'_c value.

*Joseph DiStasio**: Referring to Mr. Lord's discussion—in the first place, I know the flat slab regulations came from the work of Mr. Bertin and were written up by Mr. Peabody. Second, with regard to contention of unsafe construction caused by this rigid flat slab design, we have used the new regulations on two and a half million square feet with practically no cracking and therefore as far as I am concerned I do not subscribe to Mr. Lord's opinion as to the safety of the flat slab regulations. Furthermore I understand that Mr. Bertin has used the same regulations on other work in New York.

Chairman Richart: As a member of Committee 318, I should like to comment on these flat slab regulations. Not only have Messrs. Bertin, Di Stasio and Peabody done a great amount of work in developing these new provisions, but other writers have been active in the same field. On the Pacific Coast, Mr. H. D. Dewell has developed a similar procedure, applying frame analysis to flat slabs. I believe it is understood that the object is to provide a definite procedure for certain unusual cases of slab construction, such as slabs only one or two panels in width, which were not permitted under the old code. We retain the old provisions for the normal panel of usual proportions, but add the second or alternate method of design where there are unusual variations in spans, floor loadings, etc.

Mr. Lord has pointed out that the new procedure prescribes about 12 per cent more bending moment than the old method. Thus a penalty is applied to the use of the new method. It is well known that

†Professor of Mechanics, University of Wisconsin, Madison.
*J. DiStasio & Co., New York City.

the 1928 A. C. I. code does not call for the full theoretical bending moment for an interior slab, with all panels loaded; but uses only 72 per cent of the full statical moment. That requirement recognized the favorable conditions in such slabs, as shown by experience, analysis and tests. The unusual types covered in this code do not have this historical background, hence are not allowed as much reduction, and furthermore must be used with combinations of panel loadings which produce the maximum positive and negative moments. Hence, on two counts, the unusual panel is penalized, theoretically, to compensate for the lack of experience as to its behavior. The panel of extreme proportions will furthermore be expensive.

There is another consideration which applies to highly indeterminate structures like a slab, which have many statically possible methods of carrying load, and hence are able to adjust themselves to local conditions of over-stress. Dr. N. M. Newmark,* in a paper last year indicated that slabs with equal amounts of reinforcement might have the steel placed in a variety of positions and still produce fairly satisfactory structural behavior. This is a rather fortunate condition in view of the fact that the determination of theoretical moments in slabs is generally difficult.

R. R. Zipprodt†: I should like to say a word about the manner in which the Committee went at this matter of revising the Chapter on Flat Slabs. I am afraid there might be some implication from Mr. Lord's comments that this revision of Chapter 10 had been arrived at by the Committee in a relatively short period without too much consideration of what was actually concerned in the design. You will recall that the report of the Committee was adopted in 1936 in tentative form. I should like to call the attention of the Institute to the fact that the Chapter on Flat Slabs has had more consideration than any other single chapter throughout that period of five years, and that we submitted a report at the Chicago Convention a year ago, in such form as seemed to have arrived at the final solution. However, points were brought up at that meeting which made it necessary for us to withdraw the report for further consideration, and during the last year we have considered this chapter more than any other chapter, holding at least one two-day session in which discussion was principally centered on this Chapter. We feel now that we have the best chapter that can be written, in view of our knowledge

*N. M. Newmark, "What do we know about Concrete Slabs," *Civil Engineering* Vol. 10, No. 9, pp. 559-562, Sept. 1940.
†Secretary Committee 318.

of the factors which control design of flat slabs other than the standard flat slabs of three panels or more, which were covered by the previous Code.

A. Burton Cohen:* There is justification in the procedure of the Committee to develop factors controlling the design of flat slabs other than the standard flat slab of three panels or more. Flat slab construction, carrying railroad loading, has been used successfully in viaduct construction of single panel in width carrying two tracks, and in other bridges for grade crossing eliminations built on an angle of forty-five degrees with the center line of track, involving end or facia panels triangular in shape.

Maurice P. Van Buren†: In the Code as printed in the November JOURNAL, there were a few typographical errors. I assume in any action taken by the Convention here that those errors will be corrected.

Mr. Bradbury: Yes, they will be taken care of.

Chairman Richart: The question before us is now simply on the adoption of the report.

(The motion on the adoption of the report was then put and carried without dissent.)

Chairman Richart: It is a majority vote and the motion is carried. This adoption of the proposed new Building Regulations as published in the November JOURNAL is now subject to letter ballot of the membership after this convention. The matter will be referred to the entire membership of the Institute, of course, by letter ballot, a short period after the Convention is over. That I believe, closes the program this afternoon and the meeting will stand adjourned.

BY J. K. PEEBLES, JR.‡

*Section 709, paragraph (a), p. 104***

Although there was a considerable discussion of this paragraph in the September, 1940 *Supplement*, the paragraph does not appear to have been changed to cover the desired conditions.

As now worded, it permits walls as a support for slabs supported on four sides. This would permit brick walls which would give little restraint to the slab and, in the case of a low parapet wall above there would be practically no restraint. On the other hand, the Code prohibits the use of un-encased steel beams. This does not appear to be consistent or desirable.

There are two general cases involved, both of which are met with in practice and should be provided for in the Code. The coefficients

*Consulting Engineer, New York.
†J. Di Stasio & Co., New York.
‡Chief Engineer, Baskerville & Son, Architects, Richmond, Va.
**Section, paragraph and page references are to the November, 1940 JOURNAL.

involved cover fully the various conditions of restraint and lack of restraint and it only remains to deal with the various conditions of support. This can be accomplished by eliminating the last sentence reading as follows:—"The support for the floor slab may be walls, reinforced concrete beams, or steel beams fully encased in concrete." It does not appear to matter what type of support is used provided the correct coefficients are used to suit the particular conditions of support involved.

An un-encased steel beam would not have the slab rigidly attached to the support and the coefficients would have to be selected accordingly, but certainly should not be prohibited as a means of support.

I have serious doubts as to the advisability of allowing a slab supported on a brick wall to be considered as rigidly attached to the support. If it is to be considered that such a wall provides any rigidity, there should be definite requirements set forth as to conditions under which such restraint may be considered.

This paragraph should also stipulate that slabs, up to certain proportions of length to width (the proportions to be established as may seem best for the conditions and be set forth in the Code) and supported on four sides, be treated as such and designed accordingly. Occasions arise where, in general, the floor or roof slabs of a building are reinforced in one direction only and yet some portion of the floor or roof, due to special framing conditions are actually supported on four sides. This condition is frequently overlooked and the added supports disregarded, with the result that the slab is over-reinforced in one direction and under-reinforced in the other. One cannot make the loads act in a given direction by merely providing reinforcement in that direction and ignore the relative stiffness of the slab in the other direction. To make the code complete, I feel that such a requirement should be incorporated.

Footnote, p. 104, equation (9)

The formula: $$F_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)} 3 = 1 - r_B$$

should read: $$r_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)^3} = 1 - r_B$$

Section 1002, paragraph (a)4, p. 117

This paragraph appears to admit of a very dangerous interpretation. Under certain conditions, it might be definitely established that a given column would have considerable lateral movement or settlement and yet the amount thereof might not be reasonably determined and therefore, according to the Code, could be neglected whereas, it should be determined or a method of construction used to avoid the condition. This paragraph should be amended to avoid such an interpretation.

Section 1003, p. 118

This section and the subsequent sections in connection therewith are very desirable, but they do not stipulate what ratio of live to dead load is permissible when using the coefficients set forth. This ratio, affecting the relative maximum and minimum moments under varied conditions of loading, should be covered in the Code so that the designer can stay within these limits.

The coefficients undoubtedly were established with some condition of unbalanced loading on the various slabs and, upon this basis, it would appear highly desirable to stipulate, under the special cases covered under this section, what moment must be taken care of in the columns. This would be in line with the purpose of this and subsequent sections, to make available to the engineer information which has been previously determined, and thereby simplify the design of this type of construction.

Along the same line of simplifying the design and making available established data, it would appear highly desirable to include a table of length of bars and point of bending such as Table 5, p. 31 of the March-April, 1936 JOURNAL.

CLOSING DISCUSSION FOR THE COMMITTEE BY A. W. STEPHENS, CHAIRMAN

A study has been made of the very interesting review of the proposed Building Regulations for Reinforced Concrete presented by Mr. Peebles. The following comments are based on correspondence with those members of Committee 318 familiar with the development of the sections which were reviewed.

Section 709 paragraph (a) page 104 reads in part as follows:
"The supports for the floor slabs may be walls, reinforced concrete beams, or steel beams fully encased in concrete."

It is believed that the omission of reference to unencased steel beam supports was due to the policy of the Committee not to formulate rules involving the use of non-fireproof members. (The regulations

relating to Pipe Columns, Section 1106b, is probably the only exception to this policy.)

Mr. M. P. Van Buren writing on behalf of J. DiStasio calls attention to the note at the head of Table 2, page 105 "which provides that e shall be taken as unity when the span in direction considered is not rigidly attached to the support".

The first paragraph of Section 709 (c) should be changed to read:

"The bending moment at any section shall be determined with coefficients derived as prescribed for one way construction (Sections 701 and 702), using the following equivalent uniform load per unit length of span considered * * * etc."

Under the foregoing requirements, the extent of the restraint provided by a wall or other support must be assumed by the designer.

Concerning the suggestion that the code should require that slabs with supports on four sides must be designed according to Section 709 when the proportion of length to breadth is within some limit to be established, M. P. Van Buren writes:

"It is not believed that it is necessary to place any limitations on the proportions of length to breadth of two way slabs. Should some small part of a one way slab, due to special framing conditions, be subjected to two way action, the deflections of the structure will readjust the stresses before failure. In such cases, to avoid cracking, the designer should exercise his judgment in providing cross reinforcement and in providing for the loads to be carried by special beams."

As stated, footnote equation (9), page 104, should be changed to read:

$$r_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)^3} = 1 - r_B$$

It should be noted also that $q_A{}^x$ in equations (5) and (6) on page 106 should be changed to $q_A x$.

Concerning the comments relating to Sections 1002 to 1009 inclusive, R. L. Bertin offers the following discussion—

Section 1002, paragraph (a)4

"In general, I am of the opinion that codes, no matter how well written, cannot guard against poor judgment or ignorance. In this case I do not believe that anyone would knowingly design a flat slab structure on a yielding foundation, without making special provision for it.

Sections 1003 to 1009:

"The B. M. coefficients specified in these sections are the ones which have been in general use. Although the elastic analysis of cases for which the specified coefficients are allowed indicates that the moments for unequal loading may exceed those specified, the committee did not see fit to increase the moments beyond those which have been in general use. Hence, the omission of live to dead load ratio limitation.

"The original coefficients were established for fixed panels in which case the unbalanced loadings have no effect on the slab moments. It is, of course, correct that this assumption is realized only when large columns support the slabs, as for instance in the lower stories of high buildings.

"Regarding the moments to be taken care of in the columns, Section 1108—'Bending Moment in Columns', calls for all columns to be designed for moments resulting from loading conditions and restraint. The proposed method of designing flat slabs by elastic analysis furnishes a means of obtaining these moments.

"As to the introduction of a table giving the length of bars and points of bending, the committee felt that these limitations are best determined for each individual case."

NOTE:—If the Convention action in approving the report of Committee 318 is ratified by letter ballot, the report, as published November 1940 (with correction of editorial and typographical errors) will become available in pamphlet form as: Standard Building Regulations for Reinforced Concrete (A. C. I. 318-41).

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Concrete Performance in an Arid Climate*

By Thomas E. Stanton†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper describes studies conducted in the Imperial Valley, Imperial County, California, on the performance of portland cement concrete in an arid region where the average humidity and rainfall are low and temperatures in the summer high. A test section of pavement was constructed in May, 1929, and therefore has been in service and under observation for eleven years. Several different curing methods were used, including bituminous impervious membranes and impervious paper of different types as well as sodium silicate, and earth and water cure. In addition to the curing tests, determination was made of the effect of admixtures of volcanic ash (Puzzolanic type), diatomaceous earth, hydrated lime and calcium chloride as well as of a seven sack mix, the standard being six sacks of cement.

In general, regardless of the method of cure or the admixtures the performance of all of the test sections has been satisfactory, except for certain construction defects which cannot be attributed to any special method of curing or admixtures.

The test section is exceptionally smooth after eleven years service, the roughness as measured by the roughometer being only 24.2 in. per mile.

HISTORY AND DESCRIPTION OF TEST PROJECT

Early in 1929 considerable pressure developed on the State Highway Department to permit the use of impervious membrane curing of the bituminous cutback and emulsified asphalt types as well as impervious paper, both over and under the pavement base. Claims were being made also for puzzolanic qualities of a volcanic ash found in San Joaquin County.

*Received by the Institute Aug. 12, 1940.
†Materials and Research Engineer, California Division of Highways, Sacramento.

It was decided that if tests were made they should be under the most severe warm and dry weather conditions encountered in the State and as a section of concrete pavement was then under construction in the Imperial Valley between the towns of El Centro and Brawley, this location was selected for the test.

Contractors have difficulty getting men to work in the Imperial Valley in the summer time for which reason construction work is usually suspended about the first of June. Because the contractor was making every effort to finish the paving before the end of May it was necessary to construct and cure the test sections under the moderately hot conditions prevailing in that month instead of during the warm weather between July and September. The work was done between May 9 and May 15, when the maximum temperatures ranged from 85 to 102°F. with an average daily maximum of 96°F. and a minimum daily humidity from 7 to 30 with an average of 16 per cent.

The first nine days after placing (the standard curing period) the maximum temperatures ranged from 91° to 101°F. with an average of 96° and the minimum humidities from 16 to 32 with an average of 23 per cent. Average maximum pavement temperatures averaged well over 100° during the same period running up to as high as 121°. Atmospheric temperatures in this area frequently reach 120° in the shade. At no time during the pouring of the concrete and subsequent curing was there any rainfall and in fact, except for light rains totaling 0.22 in. in August and September, no rain of any consequence until January 1930 when there was rainfall of 0.54 in. The total rainfall during the first 20 months immediately following the construction (Table 4) amounted to only 2.25 inches.

Brawley in the Imperial Valley at the north end of the test project is 119 ft. below sea level and experiences a low average rainfall. The maximum rainfall in any of the last eleven years was 8.18 in. in 1939 (Table 4), the total rainfall from May 1929 to May 1940 amounting to 29.97 in. or an average of 2.67 in. per year with only eleven hundredths of an inch during the calendar year of 1934.

From monthly and annual rainfall figures (Table 4) it will be noted that the heaviest rainfall for the entire period occurred during September, 1939 when 6.75 inches fell in three days. It would be of interest to know just how much this above-normal precipitation had to do with the increased concrete strength as determined from cores cut eight months later in May, 1940.

The driest spell of the entire eleven years immediately preceded the five year period when but two hundredths of an inch of rain fell between May, 1933 and May, 1934, whereas the wettest year immed-

TABLE 1—FLEXURAL TESTS*

Imperial Valley Curing and Admixture Test Series

Sec. No.	Description	Modulus of Rupture			
		28 Days		6 Months	
		Side in Tension			
		Top	Bottom	Top	Bottom
		Curing Series			
1	Std. Six Sk. Conc., 8 day Water Cure	520	571	712	846
2	Cutback A. C. No. 1 180-200 sq. ft. per Gal.	496	657	672	868
3	Cutback A. C. No. 1 133 sq. ft. per Gal.	468	720	638	818
4	Emulsion No. 1 Special Method 50 sq. ft./Gal.	372	560	525	824
5	Bituminous Coated Paper	498	683	660	713
6	Heavy Absorb. Paper—Bit. Enam. on Surface	435	685	722	834
7	Sodium Silicate	332	486	609	767
8	Heavy Bldb. Paper—Jts. Cemt. with A. C.	498	563	623	815
9	Emulsion No. 2—90 sq. ft. per Gal.	462	591	635	735
10	Emulsion No. 2 on soil 30 sq. ft. per Gal.	486	668	620	670
11	Water on Surf.—Bldg. Paper under slab	540	568	606	588
23	SC-4 (60-70) Asph. Oil 70 sq. ft. per Gal.	515	782	641	925
24	SC-6 (90-95) Asph. Oil 84 sq. ft. per Gal.	598	856	783	912
		Admixture Series			
1	Std. Six Sk. Conc., 8 day Water Cure	520	571	712	846
12	Seven Sack Conc., 8 day Water Cure	511	623	781	919
13	Seven Sk. + 2% C_3C^2, 8 day Water Cure	525	619	805	809
14	Volc. Ash No. 1 (Airolite) 10%	558	533	654	788
15	Volc. Ash No. 1 (Airolite) 20%	515	588	658	690
16	Diatom. Earth No. 1 (Celite) 1.5%	544	522	617	723
17	Diatom. Earth No. 1 (Celite) 3.0%	528	502	595	828
18	Volc. Ash No. 2 (Pumicite) 30%	577	597	654	688
19	Volc. Ash No. 2 (Pumicite) 15%	...	...	653	699
20	Diatom. Earth No. 2 (Dia. Flo.) 1.5%	441	494	711	801
21	Diatom. Earth No. 2 (Dia. Flo.) 3.0%	...	...	788	956
22	Hydrated Lime 10%	587	504	731	891

*10" x 6" x 32" Beams cast in pairs in pavement. Cured under job conditions and then removed and tested at ages indicated.

iately preceded the eleventh year, 8.02 inches of rain falling between May, 1939 and May, 1940.

Notwithstanding the low rainfall, low humidity and high temperatures there has been a progressive increase in concrete strengths during the intervening years; the core compressive strengths for all concrete averaging 5617 p.s.i. at eleven years as against 3992 p.s.i. at 28 days (Table 3).

DESCRIPTION OF TESTS

The curing methods and admixtures included in the test are shown in Tables 1 and 2. The test section was the right hand or east 10-ft. lane of a pavement 20 ft. wide, laid in two 10-ft. strips. The thickness averaged 6 in. for the center four feet of each strip and then increased in thickness to nine inches at each edge. The subgrade was of fine sand from New River, average thickness three to four inches laid on an old compacted, oil-treated rock surfacing, the sand being thoroughly wet down in advance of the concrete run. The pavement was divided into panels 20 ft. long, each panel being reinforced with two marginal ½-in. steel bars. The cross joints between the panels were alternately

TABLE 2—COMPRESSIVE STRENGTH OF CORES FROM CONCRETE IN THE CURING AND ADMIXTURE TEST PAVEMENT CONSTRUCTED SOUTH OF BRAWLEY, IMPERIAL COUNTY, IN MAY 1929

Imp-26-G

Test Sec. No.	Description	Job Cyls. 28 Day Comp. Str.	Age of Concrete at Time Cores Were Cut and Tested											
			28 Days		3 Months		6 Months		1 Year		5 Years		11 Years	
			Comp Str.*	% of Std.	Comp. Str.*	% of Std.	Comp. Str.*	% of Std.	Comp. Str.*	% of Std.	Comp. Str.*	% of Std.	Com. Str.*	% of Std.
			Curing Series											
1	Std. Six Sk. Conc., 8 day Water Cure	3604	4422	100	4593	100	4721	100	5362	100	5498	100	6105	100
2	Cutback A. C. No. 1 180-200 sq. ft. per Gal.	2813	3375	76.4	3126	69.6	4042	85.6	4132	77.1	4478	81.4	5127	84.0
3	Cutback A. C. No. 2 133 sq. ft. per Gal.	3257	3840	86.9	3898	84.9	4347	92.1	4262	79.5	4864	88.5	5378	88.1
4	Emulsion No. 1 Special Method 50 sq. ft./Gal.	2895	3326	75.2	3496	76.1	3509	74.3	3763	70.2	3541	64.4	4220	69.1
5	Bituminous Coated Paper	3338	4139	93.6	4370	95.2	4637	98.2	5509	102.8	5567	101.3	5733	93.9
6	Heavy Absorb. Paper—Bit. Enam. on Surf.	2988	4311	97.5	4234	92.2	3973	84.2	5496	102.5	5570	101.3	6065	99.3
7	Sodium Silicate	2770	3466	78.3	3575	77.9	3633	77.0	4598	85.7	4590	83.5	5058	82.9
8	Heavy Bldg. Paper—Jts. Cemented with A. C.	2811	4051	91.6	4354	94.8	3996	84.6	4853	90.5	5426	98.7	6072	99.5
9	Emulsion No. 2 90 sq. ft. per Gal.	2872	4286	96.9	4559	99.3	4207	89.1	4556	85.0	5307	96.5	5604	91.8
10	Emulsion No. 2 on soil—30 sq. ft. per Gal.	3888	4172	94.4	4521	98.5	4717	99.9	5151	96.1	5597	101.8	5874	96.2
11	Water on Surface—Bldg. Paper under Slab	3332	3526	79.8	3804	82.9	3810	80.8	4335	80.9	4051	73.7	4716	77.2
23	SC-4 (60-70) Asph. Oil 70 sq. ft. per Gal.		3830	86.7	4392	95.7	4548	96.3	4723	88.1	4727	86.0	4843	79.3
24	SC-6 (90-95) Asph. Oil 84 sq. ft. per Gal.		4024	91.0	4742	99.6	5188	109.9	5122	95.6	5306	96.5	6137	100.5
		W/C Ratio	Admixture Series											
1	Std. Six Sk. Conc., 8 day Water Cure	0.596	4422	100	4593	100	4721	100	5362	100	5498	100	6105	100
12	Seven Sack Conc., 8 day Water Cure	0.579	4149	93.8	4837	105.4	4785	101.4	5487	102.3	5479	99.7	6232	102.1
13	Seven Sk. + 2% C_3C_3, 8 day Water Cure	0.583	4030	91.2	4868	106.0	4935	104.5	5581	104.1	6045	110.0	7700	126.1
14	Volc. Ash No. 1 (Airolite) 10%	0.627	4024	91.0	4430	96.5	4553	96.5	5161	96.3	4925	89.6	5363	87.8
15	Volc. Ash No. 1 (Airolite) 20%	0.632	4268	96.5	4322	94.1	4497	95.3	5150	96.1	4656	84.7	5264	86.2
16	Diatom. Earth No. 1 (Celite) 1.5%	0.681	4170	94.3	4274	93.1	4463	94.5	4762	88.8	4891	89.0	5515	90.3
17	Diatom. Earth No. 1 (Celite) 3.0%	0.716	4100	92.8	3732	81.3	4065	86.1	4307	80.3	4180	76.1	4707	77.1
18	Volc. Ash No. 2 (Pumicite) 30%	0.740	3686	83.4	4575	99.6	4517	95.7	4994	93.1	4734	86.1	5228	85.6
19	Volc. Ash No. 2 (Pumicite) 15%	0.688	4108	92.9	4721	102.8	4639	98.3	5468	102.0	4764	86.7	5248	86.0
20	Diatom. Earth No. 2 (Dia. Flo.) 1.5%	0.661	4044	91.5	4423	96.4	4474	94.8	5281	98.5	4943	89.9	5988	98.1
21	Diatom. Earth No. 2 (Dia. Flo.) 3.0%	0.620	3899	88.2	4368	95.1	4407	93.4	4745	88.5	4610	83.9	5663	92.8
22	Hydrated Lime 10%	0.620	4035	91.3	4278	93.2	4539	96.2	5208	97.1	5567	101.3	6310	103.4

*Compressive strength in pounds per square inch.

expansion and weakened plane or dummy joints, the expansion joint filler being of sponge rubber ½ in. thick.

The standard pavement mix contained six sacks of cement per cubic yard. The cement came from the Redwood City plant of the Pacific Portland Cement Co. The aggregate came from a local plant at Frick, Imperial County, and contained no shale, chert or other deleterious particles. The loss in the L. A. Rattler Test was from 16.6 per cent to 22.8 per cent at 500 revolutions and from 1.0 per cent to 5.0 per cent in the sodium sulphate soundness test.

At times the mixing water in the pipe lines reached a temperature of 118°F. At a number of places concrete temperatures were taken at both top and bottom of the slab; in the bottom by means of a thermometer in a mercury bath in a small well in the concrete and in the top by placing the thermometer bulb in contact with the pavement surface and covering with a blanket. The maximum daily variation in temperature of the top of the slab was 42°F. for the bituminous coated and 34° for the uncoated surface.

The maximum difference in temperature between the top and bottom at any one time was 20° in the pavement with black surface the top being 116° and the bottom 96°. Six hours earlier the top had been 78° and the bottom 84°, a difference of 6° in the opposite direction.

The quantity of mixing water was automatically controlled by a meter installed at the mixer; samples of concrete were taken to determine relative consistencies and field cylinders were cast to determine the relative quality of the mixtures in the several sections.

In the curing series an effort was made to maintain a constant water cement ratio of approximately 0.60 by volume. The actual average w/c ratio for the standard 6 sack mix was 0.598 and average slump 1¼ in. To maintain a slump of approximately 1½ in. in the admixture series it was necessary to increase the w/c ratio as shown in Table 2. The standard water cured sections were covered immediately after the finishing with burlap, kept wet until the following morning. The pavement was then cured for eight days by ponding.

The special treatments were compared on the basis of both compressive and flexural strengths. Cores were cut for compressive strength tests at 14 and 28 days, three and six months, and one, five and eleven years. The last cores were cut at eleven years because of lack of opportunity at the end of the ten year period.

FLEXURAL TESTS

Flexural strengths were determined from 10 x 6 x 32 in. beams taken from the center of the pavement after curing under job conditions. Beams were cast in twin molds, two pairs for each treatment. In an

TABLE 3—IMPERIAL VALLEY CURING AND ADMIXTURE TEST SERIES

Age	Average Compressive Strength of Cores from All Test Sections		
	Curing Series	Admixture Series	Average
28 Days	3905	4078	3992
3 Months	4128	4452	4290
6 Months	4257	4550	4404
1 Year	4759	5126	4943
5 Years	4963	5024	4994
11 Years	5456	5777	5617

TABLE 4—MONTHLY AND ANNUAL RAINFALL AT BRAWLEY, IMPERIAL COUNTY, BETWEEN MAY 1929 AND MAY 1940

	Year											
	1929	1930	1931	1932	1933	1934	1935	1936	1937	1938	1939	1940
Jan		0.54	....	T	0.33	0.00	0.57	0.10	0.10	0.00	0.52	T
Feb.		0.00	1.47	1.38	T	T	1.96	0.33	0.00	0.75	0.31	0.67
Mar.		1.03	0.00	0.00	0.00	0.02	0.05	0.00	0.52	0.15	T	T
Apr.		0.02	0.83	0.01	0.60	0.00	0.00	0.00	0.00	0.00	T	0.00
May	0.00	0.44	0.00	0.00	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
June	T	T	0.00	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
July	0.00	T	0.00	0.00	0.00	0.00	0.00	0.00	0.15	0.04	T	
Aug.	0.10	0.00	0.36	0.00	0.00	0.00	0.50	0.00	0.00	0.20	0.00	
Sept.	0.12	T	0.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.75	
Oct.	0.00	T	0.15	3.90	T	0.00	0.00	0.10	0.00	0.00	0.00	
Nov.	T	T	0.08	T	T	0.00	0.00	0.24	0.00	0.00	0.60	
Dec.	T	0	0.26	0.25	0.00	0.09	0.36	0.33	0.06	1.45	0.00	
Totals	0.22	2.03	3.33	5.69	1.08	0.11	3.44	1.10	0.83	2.59	8.18	0.67

Total 29.27 inches bet. May 1929 and May 1940

effort to avoid radial cracking, from each beam set, a ½-in. reinforcing bar was placed in the concrete outside of and around the beams. This precaution, however, did not prevent the radial cracking. The beams were tested at the site of the work, two cantilever breaks being made on each beam, one with the top and the other the bottom in tension. The first pair representative of each treatment were removed and broken at 28 days and the second pair at six months.

The results were rather erratic (Table 1) but certain general trends were evident. With one exception the flexural strengths, with the original top in tension, were invariably lower than with the bottom in tension, thus indicating superior curing for the underside of the slab than was given by any surface cure, including water.

The one exception was the six months break in the section with building paper on the subgrade (Sec. 11). With the exception of Section 4 this section likewise gave the lowest compression tests. The water cement ratio on this section was accidently slightly higher than the average (0.618 as compared with an average of 0.598). This higher water content probably had some slight effect on the strength.

In all cases, however, there was a substantial increase in strength at six months as compared with 28 days, even the emulsion cured section (No. 4) which gave such low breaks in the core tests, showing a substantial increase from 322 lb. at 28 days to 525 lb. at six months, although this latter figure was well below the minimum for the other sections at six months.

CURING SERIES

A number of methods of curing concrete, current in 1929, were tried out. Several were found to be substantially as effective as water cure and in one or two cases apparently superior, although it should be realized that the only direct basis of comparison is the compressive strength of cores cut at various ages. There are many factors affecting compression tests, one of the most frequent being the position and character of the coarse aggregate in the test specimen. With aggregates running as high as three inches in size and frequently with flat smooth sides in the shear planes comparatively low breaks are bound to occur with some specimens, whereas other specimens with well placed interlocking particles frequently develop remarkably high strength. If a sufficient number of specimens are available for test, erratic breaks may be largely averaged out. Cores are expensive, however, and one or two erratic breaks in a total of six cores may make the test strength above or below standard. Due allowance therefore should always be made for erratic and variable construction and test conditions, such as unavoidable variations in mix, etc., in evaluating curing methods.

On true test projects, unhampered by the urge to hurry the work which always prevails on a major construction project into which a special test is injected as a side issue, it is practicable to adjust and control the mix so as to reduce variables to a minimum. This was not possible, however, on the Imperial Valley project where the test sections aggregated less than four per cent of the entire project and the contractor was hurrying to beat the hot weather. Even so, however, the information secured is of considerable interest and, it is believed, of value.

There was a progressive increase in strength regardless of curing method. The compressive strength of the standard water cure increased from 4422 lb. at 28 days to 6105 lb. at eleven years (Table 2).

Six of the twelve alternate curing methods averaged over 90 per cent of Standard at eleven years, ranging from 91.8 per cent for an asphaltic emulsion applied at the rate of 90 sq. ft. per gal. to 100.5 per cent for a heavy asphaltic road oil (SC-6) applied at the rate of 84

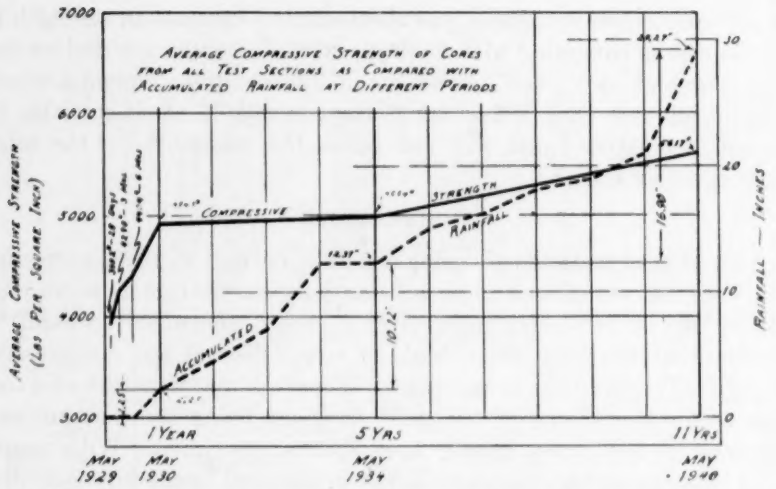

FIG. 1—IMPERIAL VALLEY CURING AND ADMIXTURE TEST

sq. ft. per gal. The impervious paper protected sections ranged in between from 93.9 per cent to 99.5 per cent Standard.

Poorest results were secured with a bituminous emulsion screeded into the surface of the green concrete (Sec. 4). At all ages this section averaged less than 85 per cent of Standard and at the end of eleven years registered but 69.9 per cent Standard. An inspection of the cores indicates that the low breaks were due to a crushing of the soft surface rather than to weak concrete below the top inch. Even so, the eleven year cores from this weakest section averaged 4220 p.s.i. and there is no evidence of pavement distress or failure except that the original surface finishing marks have for the most part been worn off and the surface therefore presents a smoother texture than the other sections.

It is of interest to note that the average increase in strength for all sections in both the curing and admixture tests between the ages of five and eleven years was relatively considerably greater than between the ages of one year and five years (Table 3 and Fig. 1).

This condition maintained not only for the average of all cores covering 24 test sections but also for 21 individual sections (Table 2). In a number of cases there was a retrogression at five years as compared with one year, followed by a decided pick-up at eleven years.

The thought naturally occurs as to what bearing the relative rainfall during these respective periods might have had on the results.

The greatest increase in strength was in the case of the seven sack mix with 2 per cent calcium chloride, the average strength increasing from 4030 lb. at 28 days to 6045 p.s.i. at five years and 7700 lb. at 11

FIG. 2 (TOP LEFT)—FORMS FOR TEST BEAMS

FIG. 3 (TOP RIGHT)—FINISHED PAVEMENT SHOWING TEST BEAMS

FIG. 4 AND 5 (BOTTOM)—SHOWING CONSISTENCY OF CONCRETE—1½-IN. SLUMP

FIG. 6 (TOP LEFT)—SPRAYING ASPHALTIC IMPERVIOUS MEMBRANE

FIG. 7 (TOP RIGHT)—SPRAYING A.C. EMULSION ON EARTH COVER

FIG. 8 (BOTTOM LEFT)—ASPHALTIC COATED PAPER ON BURLAP

FIG. 9 (BOTTOM RIGHT)—HEAVY ABSORBENT PAPER—UPPER SURFACE ASPHALT ENAMEL COATING

years with one specimen breaking at 9075 lb. and two other specimens over 8000 lb., the average at eleven years being lowered by one specimen with a large flat smooth surface in the shear plane, which notwithstanding this defect, broke at 6037 lb. The same section (No. 13) showed a consistent increase throughout the entire period. What part, if any, did the calcium chloride admixture play and was this delinquescent salt an aid to hydration in picking up some of the small amount of moisture normally present in the atmosphere?

ADMIXTURES

Several admixtures were tested, including volcanic ash from near Friant in Fresno County*, two diatomaceous earths, and hydrated lime.

(1) The volcanic ash came from two deposits several miles apart in the vicinity of Friant, Fresno County. The composition of the two deposits was essentially the same but one was a little more finely divided than the other and as there was some commercial competition between the two sources, both grades were included in the test. It was originally intended to use the same percentages of each but some misunderstanding in one case as to the required quantity, necessitated using different percentages, the final quantities being: (a) Volcanic ash No. 1, 10 per cent and 20 per cent by weight of cement; (b) Volcanic ash No. 2, 15 per cent and 30 per cent by weight of cement.

(2) The two grades of diatomaceous earth were: (a) Celite (from Lompoc, Calif.) 1½ and 3 per cent by weight of cement; (b) Dia Flo (from Carlin, Nevada) 1½ per cent and 3 per cent by weight of cement.

(3) Ten per cent hydrated lime (by weight of cement) was added to the concrete in one test section (No. 22).

In all cases the admixtures were added at the mixer to the standard concrete mix as delivered by trucks in weighed batches. The result was that there was no practical opportunity to adjust the mix accurately for constant cement content, either by weight or volume. Consequently the cement content of all admixture mixes is less than the standard 6 sacks, the exact deficiency being unknown except for a theoretical determination based on unit weights of the added materials. No effort was made to make an exact determination but due allowance must be made for this situation in analyzing test results.

Referring again to Table 2, it will be noted that the only admixture section which averaged over 100 per cent Standard at eleven years was the 10 per cent hydrated lime section (No. 22). This section showed relatively low strength at 14 days (88 per cent Standard) but had a

*JOURNAL, Amer. Concrete Inst.; Oct. 1931; *Proceedings*, Vol. 28, p. 131.
JOURNAL, Amer. Concrete Inst.; Sept.-Oct., 1934, *Proceedings*, Vol. 31, p. 33.

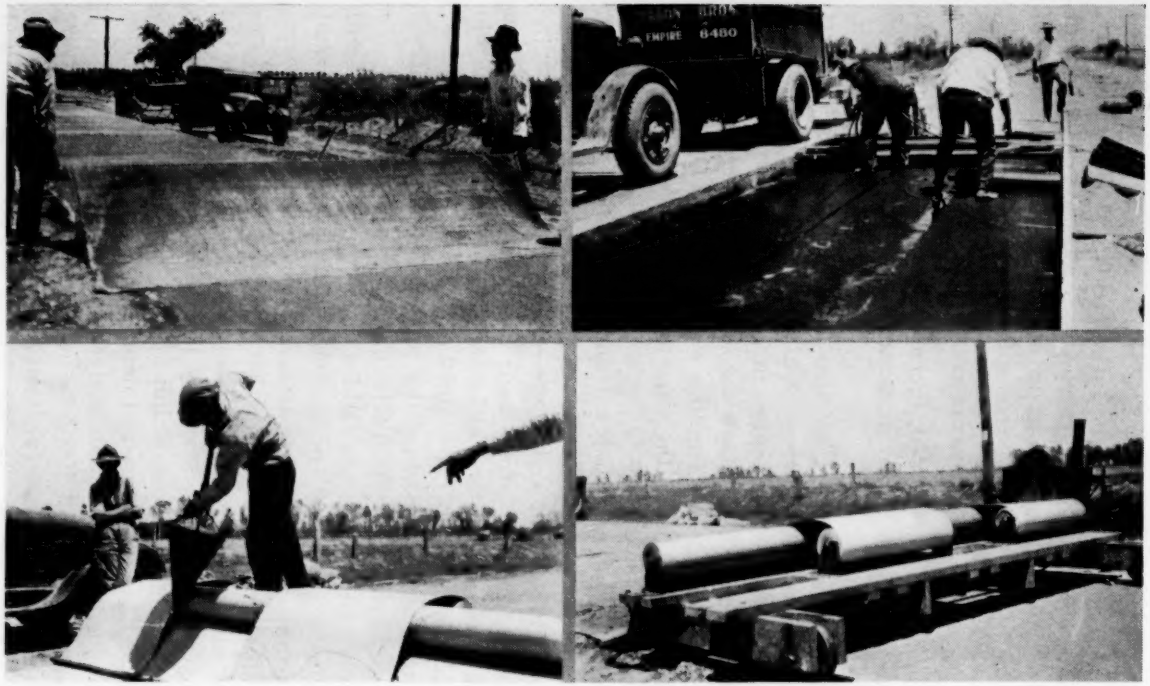

FIG. 10 (TOP LEFT)—SPREADING BURLAP ON GREEN CONCRETE

FIG. 11 (TOP RIGHT)—LAYING HEAVY BITUMINOUS COATED PAPER ON SUBGRADE

FIG. 12 (BOTTOM LEFT)—HEAVY DOUBLE THICKNESS BUILDING PAPER

FIG. 13 (BOTTOM RIGHT)—CARRIAGE FOR LAYING BUILDING PAPER

FIG. 14—CONDITION OF PAVEMENT (MAY 1940) AFTER ELEVEN YEARS

consistent and gradual increase at all ages up to eleven years reaching 101.1 per cent Standard at two years (figures not shown in Table 2).

The results with the volcanic ash were somewhat disappointing regardless of percentage used, although somewhat consistent with laboratory tests in which it was found that the strengths fell off in proportion to the reduction in cement content. It is, of course, very probable that lack of moisture retarded and may even have entirely voided any puzzolanic action of the admixture as laboratory tests on the same material in the presence of ample moisture do indicate some meritorious properties even though considerably less than claimed for it in 1929.

The average strength for all four volcanic ash sections at eleven years was 86.4 per cent of Standard, varying from 85.6 per cent for 30 per cent of the No. 2 grade to 87.8 per cent for 10 per cent of the No. 1 grade.

The results with the diatomaceous earths were erratic but in all cases less than 100 per cent Standard. In this the reduction in cement content was undoubtedly a factor, being greater in the case of the 3 per cent admixture than with 1 ½ per cent, since no adjustment of cement content was made to hold the yield constant when an admixture was added to the standard weight batch; thus the cement content was less than standard per cubic yard due to batch increase to the extent of the admixture.

PRESENT CONDITION OF PAVEMENT

Notwithstanding the fact that the traffic has averaged substantially over 3000 vehicles per day with a liberal percentage of heavy trucks the general condition of the pavement after eleven years is excellent; the only apparent distress is in sections which were insufficiently protected from drying before the water or other curing seal was applied. Such sections show many fine surface shrinkage cracks which, however, do not appear to have become appreciably worse with age and have not progressed to the point where the concrete surface has required repairs. This condition is worse on the portions of the contract outside of the test section, probably being evidence of less care on the project as a whole than on the special test portions.

The only distress of any consequence has occurred at some expansion and weakened plane joints where the joints were not constructed in an accurate vertical plane and the usual condition of an uplift with the slab on one side of the joint being forced above the general level through expansion of the concrete. A number of repairs have been required on this account. This condition has been more frequent, however, on the portions outside of the test sections. Thus, as the principal traveled way maintenance expenditures have been for repairing uplifts at joints (more numerous outside the test sections) the maintenance cost per unit of test section regardless of cure or admixture has been less than the cost per unit on the balance of the project and no one test section has required any greater expenditure than any other.

CONCLUSIONS

The results of the tests described in this paper indicate that in a hot arid region such as the Imperial Valley:

1. Progressive increase in strength with age can be expected under any of the curing methods used but that

2. The pavement surface must be protected from two rapid drying either by fogging or other suitable means from the time the concrete is poured until it is covered either with water or other suitable protective medium. Any deficiency in surface protection during the critical period will result in surface checking which may be more serious than subsequent defects caused by inadequate curing.

3. If an impervious membrane is used it must be applied in such a manner and such quantity as to insure a seal against rapid loss of moisture.*

*Table 3 shows core strengths on all impervious membrane cured sections lower than for standard water cure.

4. An impervious paper cover is quite effective if properly placed and protected.

5. Impervious paper between the pavement slab and subgrade produced unsatisfactory results under the conditions existing on the Imperial Valley project.

6. Puzzolanic type admixtures are apparently ineffective under deficient moisture conditions.

7. Calcium chloride incorporated in the mix with the mixing water is apparently effective in maintaining a higher moisture content requisite for continued hydration and higher strength.

8. Hydrated lime is effective as a producer of strength over a period of years.

ACKNOWLEDGMENTS

The investigations and tests described in this paper were conducted as a research project of the California Division of Highways of which State Highway Engineer Chas. H. Purcell is Chief. George R. Winslow, Assistant Construction Engineer, was in charge of the field part of the test, acting as the representative of both the Materials and Research and the Construction Departments. He was assisted in the field by W. E. Peck of the Laboratory staff. The work was done under the general direction of E. Q. Sullivan District Engineer, and the immediate supervision of R. C. Payne, Resident Engineer. The paving was done by the Butterfield Construction Co., subcontractors for the R. E. Hazard Contracting Co., the general contractors.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June, 1941.*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER, 1940*

Correlation Between Rapid Immersion and Standard Four Cycle per Day Freezing and Thawing Tests*

By E. R. Dawley†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Progress report of an investigation attempting to shorten the time required to measure resistance of rock to freezing and thawing. The rapid method consists of a direct immersion of the specimen in freezing solutions for ten minutes, then thawing in running water for five minutes, wiping, and returning to the freezing solution. Two solutions, Skellysolve F and mercury, were used. Tests of three kinds of rock by the standard method and the rapid method are described. Disintegration of the specimens was slightly more rapid using the rapid method. The standard method caused more damage by splitting.

The standard method of freezing and thawing rock or concrete at Kansas State College is as follows: Trays or cans which contain samples and a small quantity of water are placed in a freezing compartment and allowed two and one-half hours to freeze. They are then placed in water at 80°F. and thawed for at least 20 minutes, after which they are partially drained, and returned to the freezer. The method yields four cycles per day. The need of a faster method is obvious. For example, one of our investigations compares the durability of pastes made from various brands of cement by counting the number of cycles required to rupture a glued joint between two pieces of rock. Some rock samples cemented together last June have not separated after 800 cycles. This test requires durable rock which will outlast the cemented joint. Merely to select several kinds of rock suitable for this test may require a year of preliminary testing. The effect of changing age complicates concrete tests, since a specimen which was 28 days old when freezing and thawing started was 78 days old

*Presented at Research Committee Session, 36th Annual Convention Amer. Concrete Inst., Chicago, February 1940; subsequently released by the author for publication. (Research Committee contributions are otherwise not for publication.)

†Professor of Engineering Materials, Kansas State College, Manhattan, Kansas.

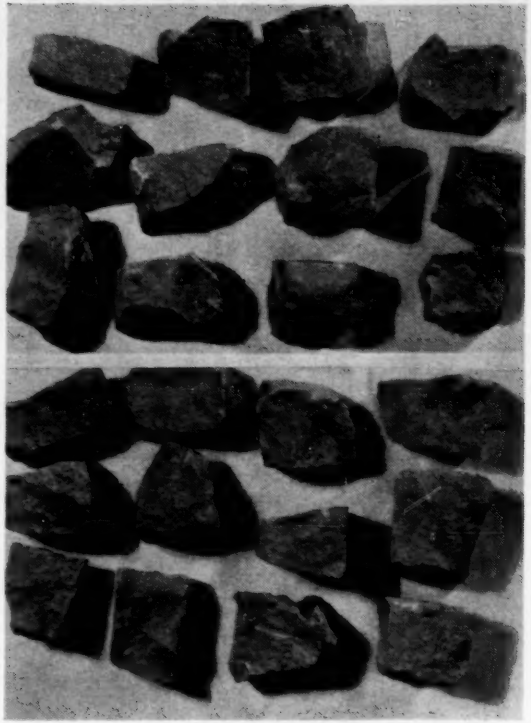

FIG. 1—BLUE RAPIDS LIMESTONE—AT TOP GROUP FOR STANDARD TEST—BELOW FOR FAST TEST

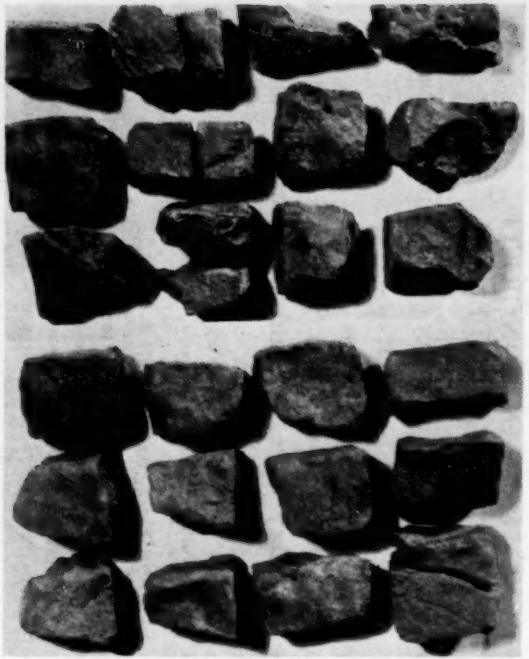

FIG. 2—TOP GROUP AFTER 38 CYCLES OF STANDARD TEST; BELOW AFTER 38 CYCLES, FAST TEST

after 200 cycles. Many specimens actually gain in strength, due to increasing age during the early freezing and thawing cycles. For these, and other reasons, a study is being made of a faster method. This report will describe three preliminary tests.

A study of our standard method brought out the following facts: (1) Heat transfer from specimens to freezer walls was largely through air, and hence, slow; (2) the water in the bottom of the trays had to be frozen before the specimens could freeze, thus, delaying freezing; (3) the damage to durable specimens by freezing and thawing, as indicated by their bullet-shaped ends, was surface damage rather than internal damage.

The first two facts indicated the possibility of increasing the rate of heat transfer by direct immersion of the saturated specimen in a liquid, and the last fact indicated that since the surface alone was affected, and since the thermal stresses have been shown to be a maximum at the surface, perhaps shorter freezing time would produce the same amount of damage. Intermittant tests have been made during the last three months using one fast freeze and thaw cycle, two different liquids, and three different kinds of rock. The ideal liquid in which to immerse the specimens has not been found, and possibly never will be found. The liquid must not freeze above —40°F., must not mix with water, and must not penetrate the specimens. The two liquids used were Skellysolve F and mercury. Skellysolve F is a volatile petroleum naptha of rather closely controlled composition. A fast cycle was arbitrarily adopted in which a water-saturated specimen was immersed in the freezing liquid for ten minutes, then thawed in running water for five minutes, and wiped or drained for one minute; then returned to the freezing solution. Three tests were run as follows: First, two of our most durable Kansas rocks, a sandstone and a limestone, were frozen and thawed at the fast and the standard rate. After applying a wiping correction, it was found that at 250 cycles the *standard* limestone sample had lost only 65 per cent as much weight as the *fast* sample frozen in Skellysolve. The sandstone lost no weight in the standard freeze. Under the fast freeze it lost 1.9 per cent, but the wiping correction accounted for all of this. The fast freeze has now been run to 775 cycles, but it will be many months before a comparison can be made with the standard method. This test indicated that the fast cycle was equal to or greater in severity than the standard.

The second test, (Fig. 1) which was run on a rather poor local limestone, in order to get quick results, compared the standard rate with the fast rate using Skellysolve. At 38 cycles, (Fig. 2) the standard freeze caused a loss in weight of 3 per cent, and the fast freeze in

Skellysolve caused a loss of 2.9 per cent. This rock was much more porous than rocks first tested, and there is some evidence that Skellysolve was absorbed into the pores. Instead of wiping the fast freeze sample as in the first test, these rocks were placed in a basket and dipped in a 70°F. Skellysolve bath to remove free water before immersing in the cold Skellysolve solution. This method of removing water appears to be superior to wiping with a cloth. However, it must be remembered that liquids like Skellysolve are inflammable.

A distinct difference was noted in the action of the fast and standard cycle on the porous rock, the fast cycle tending to remove finely divided material from the surface, while the standard method tended to break larger pieces from the specimens. (Fig. 2).

The third test was run at the fast rate on a single piece of the local limestone used in the second test, using mercury as the freezing liquid. The sample weighing 104.5 grams failed by breaking at 47 cycles, but gained 1.4 per cent in weight. There is evidence that mercury penetrated the larger pores, which would make this an expensive test on a large scale.

The use of Prestone, which has admirable thermal properties, and of tight-fitting membrane coverings which will prevent the specimen from absorbing the freezing liquid are on the program. Also, mortar specimens all having 3.27 cu. in. volume, but various shapes, and different surface areas will be used in the test. The writer will welcome additional suggestions.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June 1941.*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER 1940*

The Effect of Various Reagents on the Heat Liberation Characteristics of Portland Cement*

By L. R. Forbrich†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The temperature rise of concrete caused by the hydration of the cement and the resulting tendency to crack due to subsequent cooling, have long been recognized. In this paper a method is described of controlling the heat evolution of cements by the addition of small quantitives of active reagents. Data are presented showing the effects of a dispersing agent, an inorganic accelerator, an organic catalyst, and various combinations of these reagents, on the heat liberation characteristics of several cements. Data also are presented showing the effects on strength, durability, and shrinkage of concrete, of three combinations of the reagents which appeared to have practical value. One of these combinations was designed specifically for concrete in which heat evolution, particularly at the early ages, is important, and the other two were designed for use in ordinary concrete.

INTRODUCTION

The temperature rise of concrete due to the hydration of the cement and the resulting effects caused by cooling to its ultimate stable temperature, have long been recognized, but with the increasing number of structures of massive proportions and the growing demand for more rapid construction rates, this problem has received greater attention. Furthermore, studies made on massive structures have led to the recognition of the significance of temperature effects in less massive structures.

Use of the low-heat type of cement for the control of heat evolution in several large structures which showed relatively little cracking has prompted studies to determine the mechanism whereby the structures

*Received by the Institute January 31, 1940.
†Bessemer Limestone & Cement Co., Youngstown, Ohio; formerly with Master Builders Research Laboratory, Cleveland.

were benefitted. At first it was believed that the reduction in total heat evolved which reduced the temperature rise of the concrete, was the significant factor. Later it was believed that the slow rate of heat evolution which permitted effective artificial cooling and the improved extensibility of the concrete made with the low-heat type of cement were also responsible for the improved results. More recently it has been suggested that the temperature conditions near boundaries, particularly those beneath thick wooden forms are of special importance and may be of greater significance than temperature conditions in the interior of the mass. At surfaces the concrete is frequently subjected to rapid temperature changes which, under many conditions, are of sufficient magnitude to initiate cracks. On the other hand, in the interior of the mass, temperature drop from the maximum rise undoubtedly takes place very slowly which permits plastic flow to relieve stress, and, if there are no points of weakness, the concrete can withstand relatively large temperature drops without cracking. The fact that the temperature differences do not seem to be so severe when low-heat cements are used, suggests that, although reduced total heat evolution may be important, the slow rate of hardening and hence the slow rate of heat evolution, especially during the first few days may be of special significance.

The significance of early rates of heat liberation has not received a great deal of attention, but preliminary studies made by Carlson[1] by means of a conduction calorimeter, showed that different cements had vastly different early heat liberation characteristics. In Fig. 1 are shown the early heat liberation characteristics of four different types of portland cement; namely, a standard, high early strength, modified, and low heat cement. It was also found that the early rates were affected by many variables, of which the most significant were fineness, chemical composition, gypsum, cooling rate of cement clinker, temperature, admixtures, and certain chemicals. Evidence was found that the early rates of heat liberation may be related to setting time and early strength, as well as to temperature rise in concrete.

The inferences to be drawn from this investigation by Carlson and the results of a few exploratory tests indicated that there might be other and more satisfactory means of controlling the early heat liberation characteristics of the cement than the method of changing the composition, namely, through the addition of small amounts of active reagents. To determine the possibilities which this method offers, an investigation was undertaken to evaluate the effects, on the heat

[1] R. W. Carlson, "The Significance of Early Heat Liberation of Cement Paste," *Proceedings* 17th Annual Meeting, Highway Research Board, Dec. 1937.

liberation characteristics of cement and of two pure cement compounds, of certain reagents whose effects on the physical properties of concrete were already fairly well known. These reagents consisted of a dispersing agent, an inorganic accelerator and an organic catalyst.

Fairly recently the study of cement dispersing agents has made available considerable data. It is known that such materials will increase the number of discrete particles of cement available for hydration, that is, increase the available specific surface, which in turn leads to such effects as a reduction in water required for a given workability, increased strength, increased durability, decreased bleeding and others. The precise mechanism whereby these effects are realized is not always well understood, but it was not the purpose of this investigation to study these phenomena except in so far as they are related to heat evolution. Therefore an attempt was made to study the influence of dispersing agents on the heat evolution of cement and for this purpose a particular cement dispersing agent, calcium lignin sulphonate, was used.

Certain inorganic compounds will affect the rate of setting and hardening of cement, either to accelerate or retard. It appears obvious that such a compound which accelerates hardening will increase the early rate of heat evolution, but it seemed desirable to check this point and for this purpose a few determinations were made with calcium chloride.

It is well known that a number of organic materials will seriously impair the hardening of cement or even prevent it entirely, but it is less well known that certain other organic materials used in very small proportions will materially increase the strength of concrete. Such a material may be termed a catalyst since, in the proportions used, it is apparent that the added material cannot have a significant effect on strength by direct combination with any part of the cement. It must, therefore, react to promote some reaction of the cement in which the compound itself takes no part or at least only the part of an intermediate product. One particular group of such compounds, which may be generally described as substituted hydroxy-benzoic acids, has the rather peculiar property of increasing the rate of strength gain at the later ages (later than 3 to 7 days) without materially affecting the early strength relations. From the strength phenomena it might be inferred that such a material would not materially alter the rate of heat evolution at early ages but would increase it at later ages and would also increase total heat evolution. Since the mechanism of the reaction is still obscure it by no means follows that this is the case and, therefore, determinations of heat evolution were made with ortho-hydroxy benzoic acid, a typical member of the above chemical class.

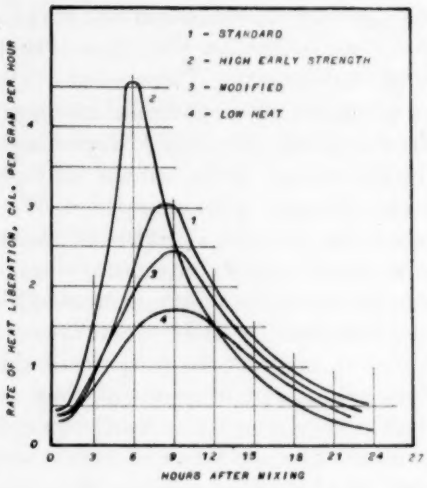


FIG. 1—TYPICAL EARLY HEAT LIBERATION RATES OF DIFFERENT TYPES OF PORTLAND CEMENT

The proportions of the reactive agents to be used in this investigation were determined by their effects on properties other than heat evolution. The dispersing agent was used up to 0.32 per cent by weight of the cement, since the plasticizing or water reducing action increases but slightly with greater proportions. The accelerator, calcium chloride, was used in amounts up to 1.0 per cent of the weight of the cement, since this proportion is effective as an accelerator and is not deleterious. The organic catalyst was used in proportions of 0.053 per cent and 0.080 per cent, which had been determined to be most efficient in increasing strength. For routine experimental convenience, these reagents were used intermixed with fly ash, which, in the small quantities used, was found to have no measurable effect on early heat liberation. A chemical analysis of the fly ash is given in the appendix.

The purpose of this investigation was primarily to determine the effects of these active reagents on the heat evolution characteristics of cement or cement compounds. With this information it seemed very probable that the different reagents could be combined in such a manner as to control the heat evolution in any desired manner or at least in a desired direction within certain limits. The latter portion of this investigation, therefore, was concerned with such an attempt, and was followed by a study of the effects of three combinations of reagents on the heats of hydration and on the principal properties of concrete.

MATERIALS

Cement

Six cements used for heat determinations were furnished by a cement research laboratory and four cements used for heat determinations and concrete tests were purchased in Cambridge, Mass. See Table D, in Appendix, for chemical analysis, fineness and specific gravity.

Aggregate

Scituate sand and gravel, purchased in Cambridge, Mass., air dried and graded. See Table A, in Appendix, for sieve analysis. Sand and crushed limestone purchased in Cleveland, Ohio.

Admixtures

The mixtures are as tabulated.

MIXTURES

Identification No.	Composition—Parts per Unit Weight			
	Dispersing Agent	Accelerator	Organic Catalyst	Pozzuolana
1	.30	0	0	.70
2	0	1.00	0	0
3	0	0	.075	.925
4	.30	.25	0	.45
5	.30	.50	0	.20
6	.30	0	.075	.625
7	.30	0	.050	.65
8	.20	0	.075	.725
9	.30	.50	.075	.125
10	.30	.25	.075	.375
11	.30	.25	.050	.400

NOTE—The admixtures were used in the amount of 1.065 per cent by weight of the cement (one pound per sack).

TEST METHODS

Heat Liberation Tests

Early rate of heat liberation—The early rates for cements were measured by means of a conduction calorimeter [1] [2] on pastes of 0.35 water-cement ratio by weight. Essentially the method consists in observing small temperature differences between the ends of a conducting metal tube of which one end is connected to a copper core inserted in the specimen and the other end maintained at about constant temperature (75°F.). Temperature differences are measured to within 0.02°C. by resistance thermometers and a special resistance test set. Since the heat is conducted away from the specimen almost as fast as it is liberated it is possible to determine the rates of heat liberation of the cements at approximately constant temperature.

Heat of hydration—Determinations were made by means of a simplified heat of solution calorimeter.[2] [3] The cement pastes were placed

[1] Carlson, R. W. and Forbrich, L. R. "Correlation of Methods for Measuring Heat of Hydration of Cement," *Ind. Eng. Chem. Anal.* Ed. 10 (7) 382-6, 1938.
[2] Lerch, Wm., *Eng. News Record*, 113, 523 (1934).

in glass vials which were stoppered, sealed, and then stored at 75°F. until age of test. Except where water-cement ratio was a variable, the water content of the paste was 0.40 by weight.

Concrete Tests

Mix data—Data on the concrete mixes used are given in Tables 3A-3C.

Procedure—Cylinders were cast from each mix according to standard methods. For compressive strength tests the specimens were either 3 x 6 in. or 6 x 12 in. cylinders. The specimens for durability and shrinkage were 3 x 6 in. cylinders equipped at each end with stainless steel gage points. Length change measurements were made by means of a Federal Dial Comparator. After the curing period the shrinkage specimens were placed in air of 50 per cent relative humidity and the specimens for durability were subjected to cycles of freezing, thawing and heating. Each cycle consisted of the following treatment: (1) Cooling the saturated specimen in water at 32°F. for two hours; (2) Freezing at 15°F. for 14 to 16 hours; (3) Heating in water at 165°F. for two hours; (4) Cooling in water at 75°F.

RESULTS OF TESTS—EARLY RATE OF HEAT LIBERATION

Tests on Pure Compounds

Since the chemical composition of cement has an important bearing upon both the rate and total amount of heat liberation, it was decided to study the effect of the dispersing agent, the accelerator, and the organic catalyst on the two pure compounds, tricalcium aluminate (C_3A) and tricalcium silicate (C_3S) which are largely responsible for heat liberation during early periods of hydration. C_3A liberates more heat per unit weight than any other major compound and this occurs almost entirely within the first 24 hours. C_3S, on the other hand, liberates heat much more slowly during the first 24 hours, but since it is present in much larger quantities it may actually contribute more heat than the C_3A.

The C_3A used in these experiments was ground with 12 per cent gypsum to a surface area of 1565 sq. cm. per gram. This amount of gypsum, which was necessary to prevent too rapid setting, corresponds approximately to the proportions per unit of C_3A that exist in cement. It is believed that the presence of the gypsum would affect mostly the time at which maximum heat generation occurs and would not influence the relative effects of the active agents. The C_3S was ground without additions to a surface area of 1620. The pure compounds used were furnished by Prof. R. W. Carlson.

In the tests described below, the reagents, except the calcium chloride, were added with the pozzuolanic material (fly ash); the amounts of reagents used, by weight of the cement or of pure compounds, were as follows: Dispersing agent, 0.32 per cent, organic catalyst, 0.08 per cent, and calcium chloride 1.0 per cent.

Tricalcium aluminate—The results of the tests made on this compound are presented graphically in Fig. 2. It is seen that the addition of the dispersing agent delayed the occurrence of the point of maximum heat liberation by about 14 hours while the organic catalyst and the accelerator, calcium chloride, shortened slightly the time to reach the maximum rate of heat liberation. Within the limits of experimental error these active agents had no appreciable effect on the maximum rate of heat evolution or on the total heat evolved during the first 24 hours.

Tricalcium silicate—The results of tests made on C_3S, Fig. 2, show that the dispersing agent greatly retarded the heat generation, at least during the first 27 hours, so that there was only a small amount of heat evolution up to that time. The organic catalyst also had a retarding effect, but it was considerably less than for the dispersing agent. It delayed the time of maximum heat generation by 8-10 hours but did not affect the total heat evolved in 27 hours. Calcium chloride had a marked accelerating effect on C_3S. It increased the maximum rate of heat generation almost 5 times, shortened slightly the time to reach this maximum, and increased the total heat evolved by more than 50 per cent during the first 24 hours.

Tests on Portland Cements

The effects of the three active compounds on the heat liberation characteristics of two standard portland cements are shown in Fig. 3. For both cements, calcium chloride had an accelerating effect as shown both by the increase in the maximum rate of heat generation and by the shortening of the time required for attainment of this maximum. The dispersing agent, on the other hand, had a retarding effect. For cement L the heat curve was flattened considerably; the maximum rate was cut almost in half so that the heat curve somewhat resembled that for a low heat cement (Fig. 1). For cement U, which has a lower C_3A content than cement L, the retarding action of the dispersing agent was greater. In this case the maximum rate of heat generation did not take place until after about 30 hours. The effects of the organic catalyst were small; in both cases, it produced some retardation.

COMBINATION OF REAGENTS

The results of tests on the three active reagents with two pure cement compounds and with two portland cements indicated that these

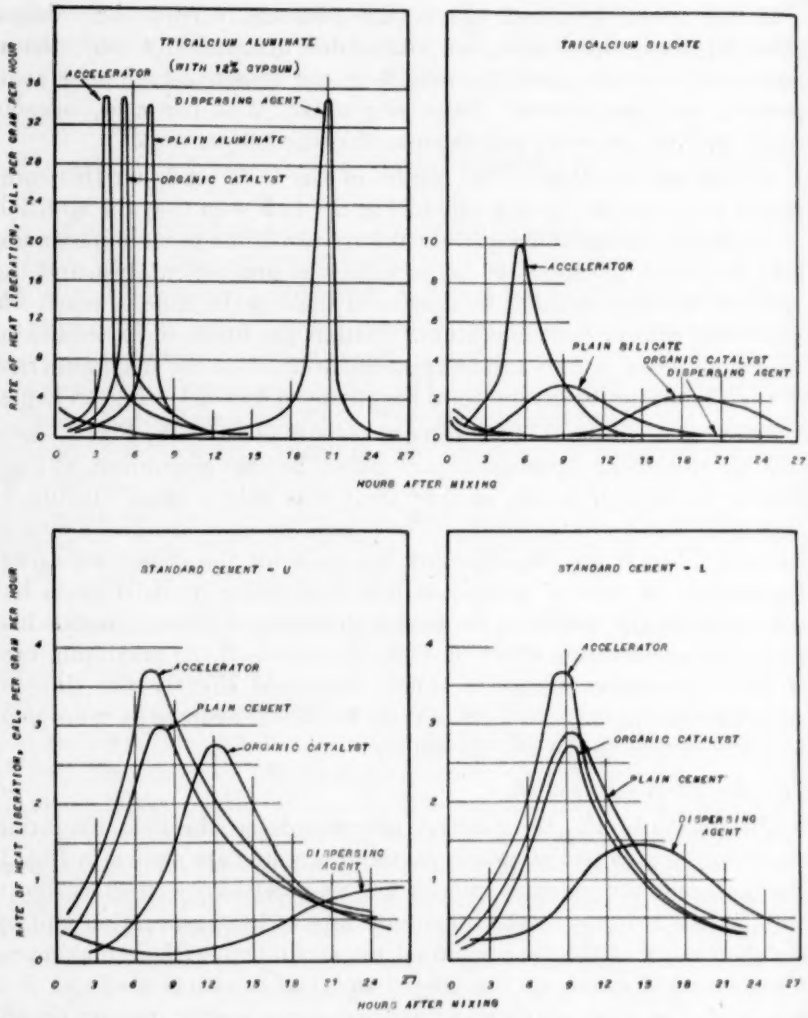
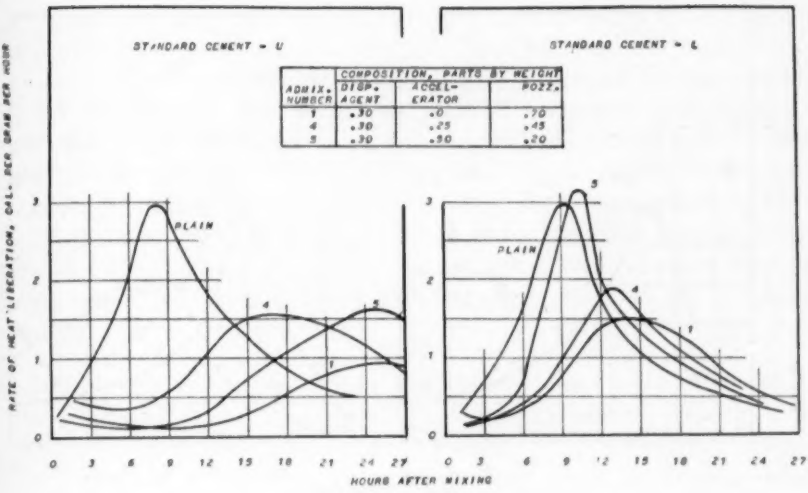


FIG. 2—EFFECT OF VARIOUS REAGENTS ON HEAT LIBERATION RATES OF TRICALCIUM ALUMINATE AND TRICALCIUM SILICATE

FIG. 3—EFFECT OF VARIOUS REAGENTS AND COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF TWO DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

reagents had definite influences on the early rates of heat evolution. The dispersing agent had a retarding effect, the calcium chloride an accelerating effect, and the organic catalyst apparently only a small retarding effect. From these results it seemed that it would be possible



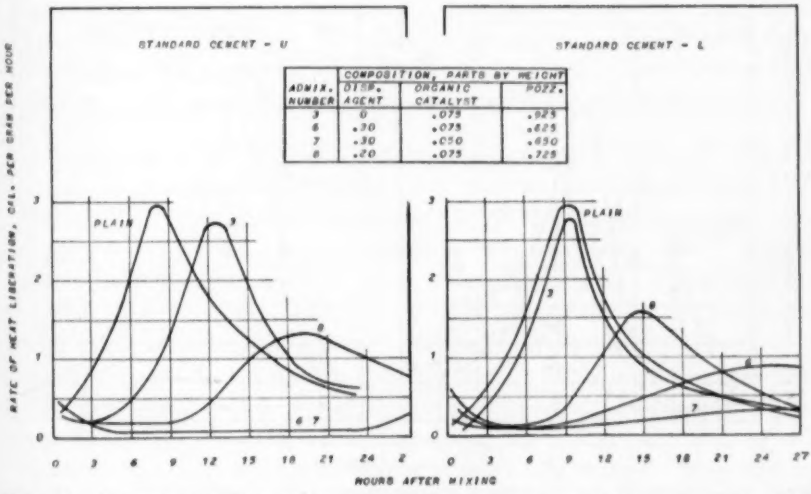


FIG. 4—EFFECT OF VARIOUS REAGENTS AND COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF TWO DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

FIG. 5—EFFECT OF VARIOUS REAGENTS AND COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF TWO DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

to control the heat liberation characteristics of cement to any desired degree by the use of varying combinations of the three reagents. With this objective the effects of eight different combinations of the three

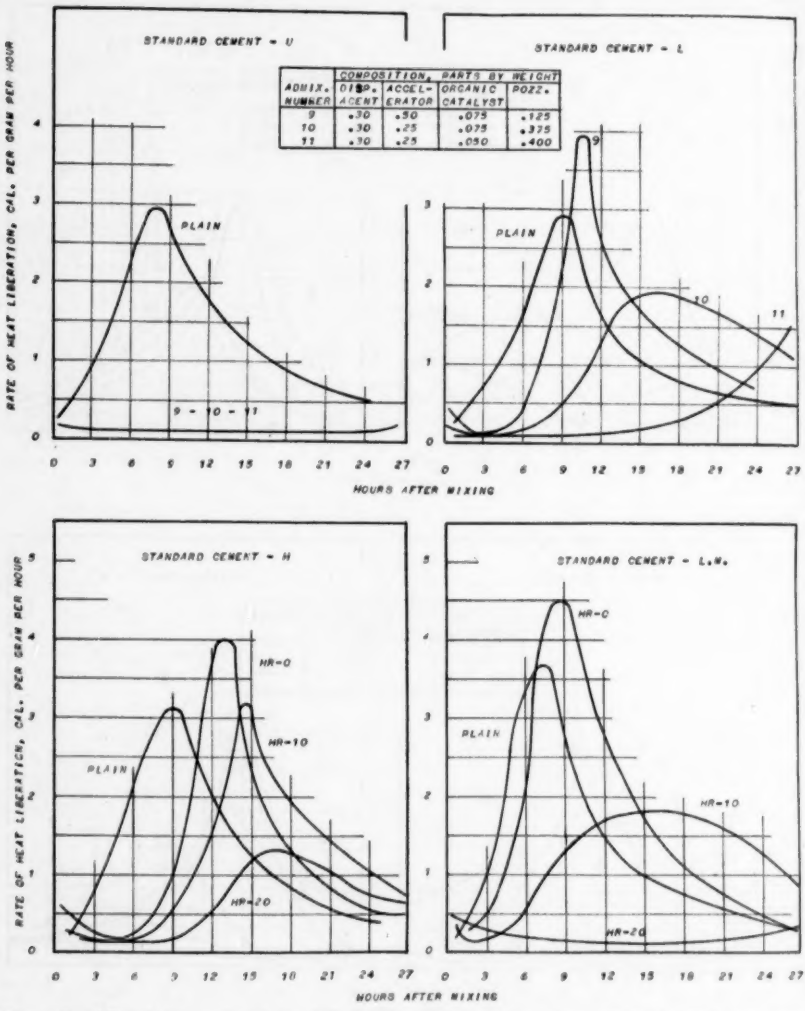


FIG. 6—EFFECT OF VARIOUS REAGENTS AND COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF TWO DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

FIG. 7—EFFECT OF THREE COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

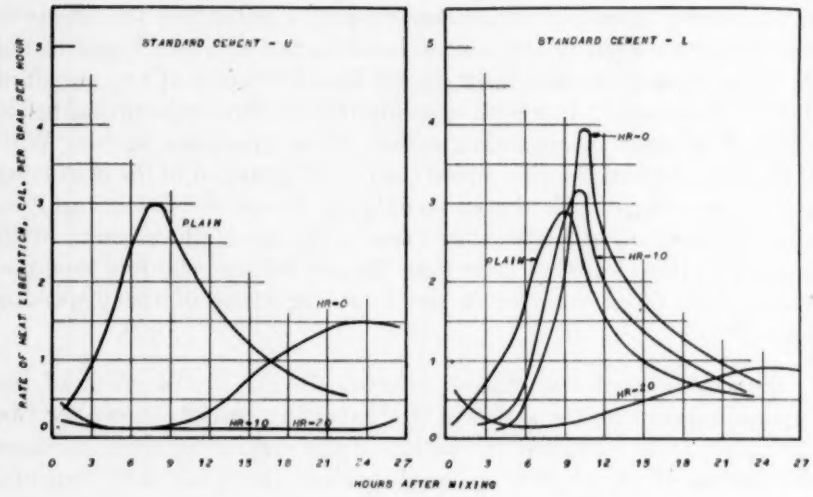
reagents were studied on several different portland cements. The combinations, the composition of which are given under Materials and on the diagrams, were added in the amount of 1.065 per cent by weight of the cement.

Dispersing agent and accelerator—In Fig. 4 are shown the effects of the dispersing agent alone, and in combination with two quantities of calcium chloride, on the early rate of heat liberation of two standard portland cements. For both cements the accelerator compensates to different degrees the retarding action of the dispersing agent. With 0.50 parts calcium chloride added the retarding action of the dispersing agents on cement L is almost completely compensated (curve 5), so that the heat curve is about the same as for the plain cement. With cement U, it is clear that more than .50 part calcium chloride would be required to offset completely the retarding effect of the dispersing agent.

Dispersing agent and organic catalyst—In Fig. 5 the effect of the organic catalyst on the action of the dispersing agent is shown for two cements. It is seen that the action of the organic catalyst increases the extent of retardation of the dispersing agent for both cements. (Curves 6 and 7). On cement U, the combined effects of dispersing agent (0.30 parts) and organic catalyst caused extreme retardation during the first 24 hours. By reducing the quantity of dispersing agent, retardation, as would be expected, was lessened (curve 6) but small differences in amounts of organic catalyst had a negligible effect.

Dispersing agent, organic catalyst, and accelerator—Fig. 6 shows the effects of combinations of the three reagents on two cements. For cement U, the retardation produced by the dispersing agent and organic catalyst was so great that the effects of additions of different amounts of calcium chloride were indistinguishable. With cement L the effects of the combinations are more clearly shown. The effect of 0.50 parts of accelerator was to influence the retarding action of the other reagents to the extent that the maximum rate of heat evolution was increased beyond that of the plain cement although the beginning of rapid heat evolution was delayed slightly. With a smaller amount of accelerator the retarding effects of the other reagents was only partially compensated as would be expected. The curves obtained for the combinations with two different proportions of organic catalyst do not seem to be consistent with other data obtained.

Name	Composition—Parts by Weight			
	Dispersing Agent	Accelerator	Organic Catalyst	Pozzuolana
HR-0	.30	.50	0	.20
HR-10	.30	.50	.075	.125
HR-20	.30	0	.075	.625



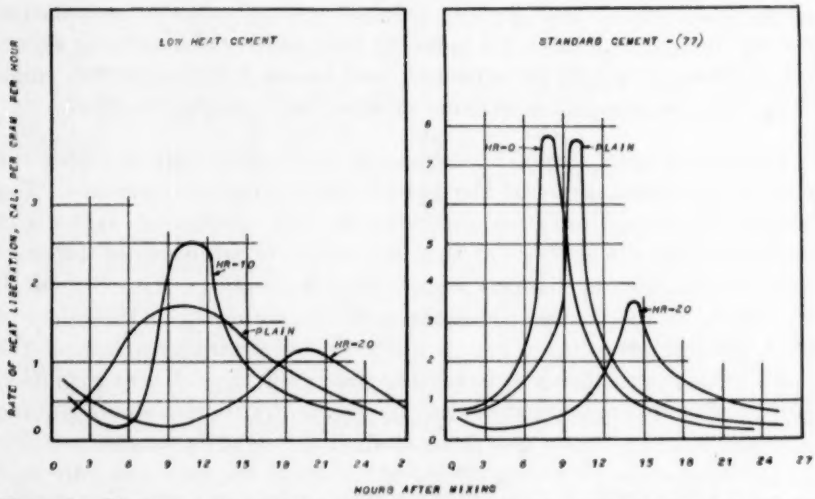


FIG. 8—EFFECT OF THREE COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF DIFFERENT BRANDS OF STANDARD (NORMAL) PORTLAND CEMENT

FIG. 9—EFFECT OF THREE COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF DIFFERENT TYPES OF PORTLAND CEMENT

HR-0, HR-10, HR-20

The results of the tests on the effect of different combinations of accelerator, dispersing agent, and organic catalyst, prompted further detailed studies on three combinations which showed evidence of

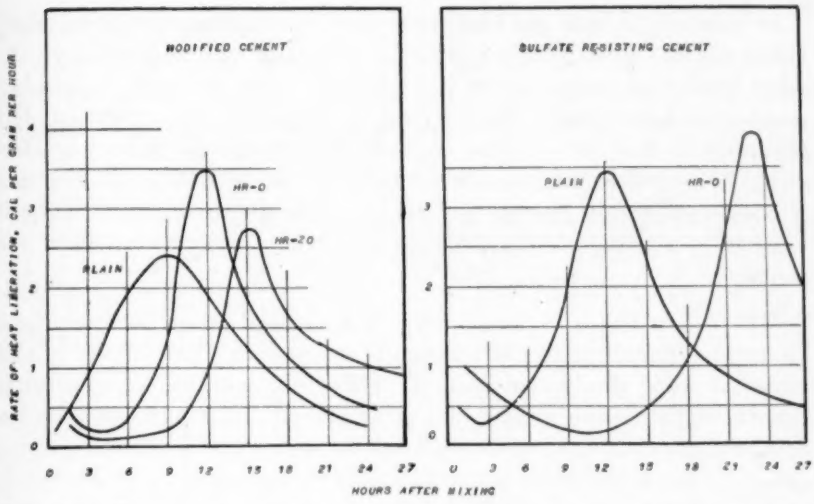


FIG. 10—EFFECT OF THREE COMBINATIONS OF REAGENTS ON HEAT LIBERATION RATES OF DIFFERENT TYPES OF PORTLAND CEMENT

practical value for use in concrete. The three combinations selected were named, HR-0, HR-10, HR-20, and had compositions as tabluated. HR-0 was designed for ordinary construction work where heat evolution is not of importance but where the effects of a dispersing agent on the other properties of concrete and normal hardening rates are desired. HR-10 was proportioned for similar application where the effect of the organic catalyst in producing increased strength at the later ages as well as the effects of the dispersing agent are important. HR-20 was designed for work in which heat evolution, particularly at the early ages, is the primary consideration.

HR-0—In Fig. 7 to 10 the effect of HR-0 on the early rates of heat liberation of 7 different cements are presented. For the standard portland cements, Fig. 7, 8, and 9, HR-0 generally delayed the time at which rapid heat evolution occurred; for one cement, (Fig. 9) however, there was a small accelerating effect. The influence of this material on total heat liberated up to one day was small; in one case only was there any significant effect, and that was for cement L.M., which showed an increase of about 5-7 calories. In Fig. 10 the effect of HR-0 on a modified and on a sulfate resisting cement are shown. As for the standard cements, the start of rapid heat evolution was delayed, but the maximum rates in both cases were increased so that in 36 hours (27-36 hours not shown), the total heat evolved was not materially affected.

In general, it was evident, that from a practical point of view, HR-0 did not greatly affect the heat liberation characteristics or the early hardening properties of the cements. In a few cases somewhat more retardation than what would be desirable was observed, but this may be readily overcome by increasing the proportion of accelerator. HR-0, therefore, would be suitable for ordinary construction purposes and would provide the benefits of the dispersing agent without materially altering the hardening rate or heat characteristics of the concrete.

HR-10—In Fig. 7, 8, and 9, the effects of HR-10 on the early heat liberation characteristics of 5 cements are shown. The effects of this material were similar to those of HR-0, except that in general it caused slightly more retardation. For cement U (Fig. 8), which contains 6.5 per cent C_3A, somewhat less than that found in most normal cements, the retardation caused by HR-10 appeared to be excessive.

HR-20—In Fig. 7 to 10 the early heat curves for HR-20 with several different types of cement are shown. The effect of this material was consistently to delay the time to reach maximum heat generation and to reduce both this maximum rate and the total heat evolved during the first 24 hours.

The test data show that the use of HR-20 will accomplish its intended purpose of decreasing heat evolution during the first 24 hours with any type of portland cement. For some of the cements, it might be necessary to modify the composition of HR-20 to make it of practical value. The fact that HR-20 apparently retarded the hydration process of the cements might indicate that it would also impair early strength. However, concrete tests, the results of which will be discussed later, showed that this was not the case.

The early heat liberation tests with HR-0, HR-10, and HR-20 showed that within limits, the three active reagents can be combined so as to control the heat liberation characteristics of cement as desired, but that the influence of the reagents varied with different cements. An attempt was made to determine what constituents in cement were responsible for these variations, but this was not successful. The data indicate, nevertheless, that cements low in C_3A and possibly high in C_3S, are the ones retarded to the greatest degree by the dispersing agent and the organic catalyst. The influence of minor constituents, may, on the other hand, have more influence than would ordinarily be expected. It might be necessary to adjust slightly the compositions of materials HR-10 and HR-20, for use with some cements.

Heat of Hydration

HR-0—The effect of this material on the heat of hydration of 7 different cements is shown in Table 1A. It will be seen that HR-0 increased slightly the heat of hydration of most cements at 3 days; the

TABLE 1-A—EFFECT OF HR-0 ON HEAT OF HYDRATION

Cement	Admixture	Heat of Hydration—Cal. per Gram of Cement			
		3 Days	7 Days	28 Days	110 Days
Standard L.M.	Plain	70.3	85.4	103.5	....
" "	HR-0	71.3	88.0	99.2	....
Standard H	Plain	64.8	80.7	91.3	....
" "	HR-0	64.8	80.7	90.9	....
Standard (77)	Plain	65.3	77.2	92.2	....
" "	HR-0	67.9	77.2	92.8	....
Standard (91)	Plain	77.5	84.8	97.1	....
" "	HR-0	79.7	90.1	99.0	....
High Early	Plain	71.7	84.4	91.8	....
" "	HR-0	75.1	84.9	92.7	....
Modified	Plain	53.5	64.8	77.1	....
"	HR-0	56.7	65.8	76.0	....
Sulphate	Plain	56.7	69.2	77.3	
Resisting	HR-0	60.2	71.9	78.8	....

TABLE 1-B—EFFECT OF HR-10 ON HEAT OF HYDRATION

Standard L.M.	Plain	70.3	85.4	103.5	112.5
" "	HR-10	66.6	83.2	97.2	106.5
Standard H	Plain	64.8	80.7	91.3	98.6
" "	HR-10	61.6	83.9	91.5	101.0
High Early	Plain	71.7	84.4	91.8	97.4
" "	HR-10	72.3	85.9	93.0	104.0

TABLE 1-C—EFFECT OF HR-20 ON HEAT OF HYDRATION

Cement	Admixture	Heat of Hydration—Cal. per Gram of Cement			
		3 Days	7 Days	28 Days	110 Days
Standard L.M.	Plain	70.3	85.4	103.5	112.5
" "	HR-20	49.4	78.3	103.5	108.9
Standard H	Plain	64.8	80.7	91.3	98.6
" "	HR-20	63.3	83.2	97.7	111.0
Standard L	Plain	81.6	92.6	102.0	107.2*
" "	HR-20	35.6	77.9	104.8	109.8*
Standard (77)	Plain	65.3	77.2	92.2	97.9
" "	HR-20	58.1	76.1	103.8	104.5
Standard (91)	Plain	77.5	84.8	97.1	103.2
" "	HR-20	61.3	90.0	109.5	112.8
Modified	Plain	53.5	64.8	77.1	91.3
"	HR-20	52.5	61.3	75.4	94.1
Sulphate	Plain	56.7	69.2	77.3	85.9
Resisting	HR-20	39.9	60.7	83.7	88.2

*60 Days.

NOTE—HR-0 added in amount of 2.13 per cent; HR-10 and HR-20, 1.065 per cent by weight of cement. Water-cement ratio 0.40 by weight. Curing temperature, 75°F.

average increase for all the cements was 2.3 calories per gram. At the age of 7 days there was still a small increase due to HR-0 (average of 1.7 calories), but at 28 days no consistent differences were found. In general, it can be said that HR-0 slightly increased the heat of hydration at the early ages but had no effect at the later ages.

HR-10—The influence of this material on the heat of hydration of 3 cements is given in Table 1-B. The data are limited but since the observed differences are small, it appears that the material behaves in about the same manner as HR-0, except that there is some indication of a small retarding effect at 3 days rather than a small accelerating effect.

HR-20—In Table 1A data showing the effect of HR-20 on the heat of hydration of 7 cements are presented. At the age of 3 days, HR-20 reduced the heat of hydration for all cements. For two cements this reduction amounted to one or two calories but for the others it varied from 10 to 50 per cent of the heat liberated by the plain cement. At 7 days the heat of hydration was reduced for 5 cements, and at 28 days only one cement showed a reduction in heat of hydration due to HR-20; the others showed increases as much as 12 per cent of the total heat evolved by the plain cement. At 110 days most of the cements showed increased heat of hydration due to HR-20 but the increases were smaller than at 28 days.

Water-cement ratio—In Table 2 data are presented showing the effect of water-cement ratio on the heat of hydration of two cement pastes with and without HR-0. At 3 days the effect of water-cement ratio is small but at 7 and 28 days it was significant. At 28 days for an increase in W/C of 0.10 there is an increase in heat of hydration of 4-6 calories for the normal cement and 2-4 calories for the modified cement. This applies to the plain cement pastes and to pastes with

TABLE 2—EFFECT OF WATER-CEMENT RATIO AND HR-0 ON HEAT OF HYDRATION

Portland Cement—L.M.

Water Cement Ratio by Weight	Heat of Hydration—Calories per Gram of Cement					
	3 Days		7 Days		28 Days	
	Plain	HR-0	Plain	HR-0	Plain	HR-0
0.40	70.3	71.3	85.4	88.0	103.5	99.2
0.50	70.3	74.9	91.7	90.3	108.4	107.3
0.60	70.8	72.7	94.6	92.1	116.0	111.0
Modified Cement						
0.40	53.5	56.7	64.8	65.8	77.1	76.0
0.50	53.4	58.1	67.0	68.3	81.2	78.3
0.60	54.5	58.6	66.1	70.8	81.5	82.0

HR-0 as well. These effects of water-cement ratio were smaller, (about one-half) than those reported by some previous investigators.[4] [5]

CONCRETE TESTS

Compressive Strength

In Table 3 data are presented showing the influence of the three combinations, HR-0, HR-10 and HR-20 on the composition and compressive strength of concrete. The mixes used varied from 3¾ to 6½ sacks of cement per cubic yard.

Although there were some inconsistencies in the strength data, the addition of the three materials generally produced increases in strength corresponding approximately to the reduction in water-cement ration permitted by their use. In these tests the average reduction in water-cement ratio for all the mixes was 13 per cent, but it is clear that the water requirement for a given consistency depends on many variables such as type of mixing, gradation of aggregate, design of mix and others.

[4]Kelly, J. W., JOURNAL Amer. Concrete Inst., *Proceedings* Vol. 34, p. 573, June 1938.
[5]Davis, R. E., "Final Report on Cement Investigations for Boulder Dam," U. S. Bureau of Reclamation, Denver, 1934.

TABLE 3-A—PHYSICAL PROPERTIES OF CONCRETES

Ref. No.	Admix.	Comp. Cement	Fresh Concrete Lb./cu. yd.			Slump In.	Density Fresh Concrete Lb./cu.ft.	Compressive Strength Lb./sq. in.			
			Agg.	Water (Net)	Admix.			3 Day	7 Day	28 Day	90 Day
					Standard Portland Cement—H						
A-1	Plain	529	3180	269	0	2.7	148.7	2470	3030	4040	4830
A-2	HR-20	529	3190	247	5.63	2.5	147.9	2670	3400	4530	5440
A-3	HR-10	502	3030	229	5.35	2.5	145.9	2790	3760	4750	5550
A-4	HR-0	507	3050	230	10.80	2.7	146.5	3050	3630	5050	5520
B-1	Plain	432	3260	270	0	2.7	146.9	1740	2120	2990	3220
B-2	HR-20	427	3210	233	4.55	2.7	143.8	1730	2430	3480	4030
B-3	HR-10	422	3170	234	4.49	2.7	142.0	1850	2430	2720	3780
B-4	HR-0	422	3190	239	8.98	2.6	142.9	1850	2420	3230	3440
C-1	Plain	362	3290	270	0	2.7	144.9	1180	1720	2320	2770
C-2	HR-20	356	3250	230	3.79	2.7	142.3	1310	1830	2610	3170
C-3	HR-10	351	3220	237	3.74	2.5	140.3	1220	1860	2380	2740
C-4	HR-0	356	3270	239	7.60	2.5	142.8	1250	1840	2360	2960
D-1	Plain	482	3220	274	0	2.0	.....	....	....	2670	2080	....
D-2	HR-0	482	3220	245	10.25	1.8	.....	....	....	3700	3030	....
					Standard Portland Cement—L						
A-5	Plain	534	3210	275	0	2.7	150.5	2860	3760	4320	5810
A-6	HR-20	529	3200	246	5.63	2.5	148.5	2860	4090	4530	6200
B-5	Plain	437	3260	267	0	2.7	147.0	2020	2460	3640	4580
B-6	HR-20	427	3210	230	4.55	3.0	143.7	1980	2860	3770	4620
C-5	Plain	362	3270	270	0	2.5	144.8	1020	1510	2080	2540
C-6	HR-20	351	3180	228	3.74	2.7	139.7	1220	2040	2720	3240

Mix compositions computed from volumes and weights of fresh concrete.
Compressive strengths are for 3x6 inch cylinders cured 1 day in molds then in moist air at 75°F.; each value represents average of 3 tests.
D-1 and D-2 mixed by hand, all others mixed in a Lancerick open tub laboratory mixer.

Since HR-0 and HR-10 were designed to have no appreciable effect on the early hardening properties it would be expected that the early strength increases would be in accord with the reduction in water-cement ratio. In the case of HR-10 which contains the organic catalyst effective beyond 7 days, it would be expected that the increase in strength at the later ages would be somewhat greater than that which would be obtained by the reduction in water-cement ratio permitted. These effects, in general, are realized as illustrated by the data in Table 3A-3C.

In the case of HR-20 which showed a retarding effect on the early rate of heat evolution of neat cements, the compressive strength of the concrete was increased to a small degree at the early ages (3 days). At the intermediate ages, the strength increases were in accord with

TABLE 3-B—PHYSICAL PROPERTIES OF CONCRETE

Admixture	Cement Content Lb. per cu. yd.	W/C (Total) Gal. per Sack	Slump In.	Compressive Strength—Lb./sq. in.		
				3 Days	7 Days	28 Days
Plain	375	10.1	3.3	....	1770	2495
HR-20	375	8.4	2.9	1660	2345	3200
HR-10	375	8.2	2.9	....	2420	3570
HR-0	375	8.4	2.8	....	2325	3090
Plain	450	8.1	3.3	....	2255	3120
HR-20	450	6.9	3.3	2430	3340	4105
HR-10	450	6.8	3.6	....	3240	4265
HR-0	450	7.1	3.5	....	3065	3575
Plain	525	7.0	2.9	....	3010	3880
HR-20	525	6.1	3.1	2650	3850	4600
HR-10	525	6.0	3.6	....	3870	5080
HR-0	525	6.3	3.3	....	3585	4460
Plain	600	6.1	3.0	....	3405	4480
HR-20	600	5.2	2.6	2990	4695	5400
HR-10	600	5.3	2.9	.....	4600	5590
HR-0	600	5.5	2.5	....	4460	5330

Tests made at Master Builders Research Laboratory, Cleveland, Ohio, using normal portland cement and aggregate purchased in Cleveland. Mixing by machine.

Compressive strengths are for 6 x 12 inch cylinders cured one day in molds then in moist air at 70°F., each value represents average of six tests.

TABLE 3-C—PHYSICAL PROPERTIES OF CONCRETE

Admixture	Comp. Fresh Concrete Lb./cu. yd.					Slump In.	Compressive Strength Lbs. per sq. in.			
	Cement	Sand	Stone	Water	Admix.		1 Day	3 Days	7 Days	28 Days
Standard Portland Cement—L										
Plain	362	1400	1890	289	0	1.8	320	1260	....	3670
HR-20	362	1400	1890	259	3.85	0.8	360	1480	....	4400
Plain	529	1300	1880	294	0	1.8	800	3040	....	5240
HR-20	529	1300	1880	274	5.64	2.0	610	3400	....	6240
Modified Cement—S										
Plain	362	1400	1890	299	0	2.8	190	870	1460	3100
HR-20	362	1400	1890	262	3.85	1.8	210	1260	2200	4280

Compressive strengths are for 6 x 12 inch cylinders cured 1 day in molds at 70°F.; each value represents the average of 3 tests.

Tests made at Master Builders Research Laboratory, Cleveland, O.

the reduction in water-cement ratio, and at the later ages the increases were slightly greater than would be expected from the water reduction. It is apparent that at the early ages the retarding effect of HR-20 was largely or entirely offset by the physical effect of the reduction in water-cement ratio, while at the later ages there is some evidence that the organic catalyst had a beneficial influence.

Strength-heat relations—The strength-heat ratio expressed as the ratio of compressive strength to heat of hydration is in some respects a measure of the efficiency of a cement for mass concrete construction. The strength-heat ratio has been shown by previous investigators to be increased by such factors as reduction in water-cement ratio, increased fineness of cement, decreased C_3A content and increased C_2S content. The present studies have shown that strength-heat relations may be definitely influenced by the addition of combinations of certain reagents.

The influence of HR-20 on the strength-heat ratio is shown in the following tabulation. The strength data were taken from the results of concrete tests for the leanest mixes made with standard cement L (Table 3-A); heat of hydration data were taken from tests made on neat cements (Table 1-C).

Age Days	Strength-Heat Ratio Lbs. per sq. in. per cal. per gram	
	Plain	HR-20
3	12	34
7	16	26
28	20	26
90	24	30

At all ages the strength-heat ratio is markedly increased by HR-20; at the age of three days the ratio is almost three times that for the plain concrete, but the difference is less at the later ages.

The results of tests on the effect of the three combinations of active reagents on the compressive strength of concrete and on heat of hydration have brought out a further interesting relation. Since the reduction in water-cement ratio permitted by the use of HR-0, HR-10, and HR-20 reduced heat evolution (except HR-0 at 3 and 7 days and HR-20 at 28 days and beyond) a reduction in heat evolution with an actual increase in strength is obtained. It follows from this that for a given strength the heat evolution can be decreased materially by the use of these materials. This relation, therefore, affords a means of controlling total heat evolution as well as the rate.

Durability

In Table B in the Appendix the results of freezing and thawing tests on concrete mixes containing HR-0 and HR-20 are given. Although it was not the purpose of this investigation to determine the effects of these materials on durability, but to study heat evolution, nevertheless, it seemed desirable to make a limited number of tests on this important property. The data show that the durability of the concrete is enhanced by the use of these materials as would be expected from the reduction in water-cement ratio and other characteristics which they are known to have.

Other Properties

A limited number of tests on other properties of concrete were made and these will be briefly mentioned. In Table C, Appendix, the effects of HR-0 and HR-20 on drying shrinkage are shown. HR-0 appears to reduce the shrinkage slightly and HR-20 to increase it slightly.

The addition of the three combinations of materials had some effect on the unit weight of the concrete. This amounted to an average reduction of 2.0 per cent with a maximum reduction of 3.5 per cent. See Table 3.

The use of the three materials showed a definite tendency to reduce bleeding or water-gain. Although no quantitative measurements were made, reduced water-gain was readily noticeable in all mixes, especially in the leaner mixes.

SUMMARY

Tests were made to determine the effects of three active reagents on the early rates of heat liberation of cement and two pure compounds, C_3A and C_3S, and of three combinations of these reagents on: (1) early rate of heat liberation, (2) total heat liberation and (3) some of the properties of concrete.

(1) The dispersing agent retarded the early rate of heat liberation of cement and decreased the total heat evolved up to 24 hours; it delayed the time of rapid heat evolution for C_3A, and greatly retarded the heat liberation of C_3S.

(2) The organic catalyst had a slight retarding effect on the early heat liberation of cement and C_3S. With C_3A, it had a slight accelerating effect.

(3) The accelerator, calcium chloride, increased the maximum rate and total amount of heat liberated up to 24 hours. It had little effect on C_3A, but increased the rate and total amount of heat liberated by C_3S.

(4) The combination of dispersing agent and accelerator, HR-0, designed for ordinary construction purposes, in general, delayed the time of rapid heat evolution, increased slightly the total heat evolved up to 7 days and had no appreciable effect at 28 days. When used in concrete it reduced the water-cement ratio for a given workability, increased strength correspondingly and improved durability with respect to freezing and thawing.

(5) The combination of dispersing agent, accelerator and organic catalyst, HR-10 also designed for ordinary construction purposes, generally retarded heat liberation up to 3 days but had little effect thereafter. In concrete it has about the same effect as HR-0 except that at later ages there was evidence of greater increases in strength.

(6) The combination of dispersing agent and organic catalyst, HR-20, designed for work where heat evolution is important, greatly reduced the early rate of heat liberation but increased the total heat liberated at 28 and 110 days. In concrete, the strength at the very early ages was not impaired by HR-20 and at the later ages the strength was increased approximately in accord with the reduction in water-cement ratio permitted by its use.

It has been found in this study that it is possible to control the early rate of heat liberation of cement in almost any desired direction by the use of suitable combinations of active reagents. In a subsequent article it is proposed to show the influence of HR-20, which was designed to reduce early heat liberation of cement, on the temperature rise in concrete and its significance in reducing cracking.

ACKNOWLEDGMENT

This work was carried out in the Concrete Research Laboratory of the Massachusetts Institute of Technology, which is under the direction of Prof. Roy W. Carlson to whom thanks are due for his help and suggestions during the course of the work and for supplying the pure cement compounds. The writer is further indebted to Dr. E. W. Scripture, Jr., Director of Research, of The Master Builders Co., who initiated this work and supervised the preparation of this paper.

(*see Appendix p. 182*)

APPENDIX

TABLE A—SIEVE ANALYSIS OF SCITUATE AGGREGATE USED IN CONCRETE TESTS

Per Cent Retained

Sieve Opening	Mix A (1—6.0)		Mix B (1—7.5)		Mix C (1—9.0)		Mix D (1—6.7)	
	Ind.	Cum.	Ind.	Cum.	Ind.	Cum.	Ind.	Cum.
¾	0	0	0	0	0	0	0	0
¾	32	32	31	31	30	30	26	26
4	24	56	23	54	22	52	20	46
8	9	65	9	63	9	61	16	62
14	5	70	6	69	7	68	12	74
28	13	83	14	83	15	83	9	83
48	8	91	8	91	8	91	7	90
100	7	98	7	98	7	98	6	96
Pan	2	100	2	100	2	100	4	100
F.M.		4.95		4.89		4.83		4.87

TABLE B—EFFECT OF HR-0 AND HR-20 ON DURABILITY OF CONCRETE

HR-0—Mix D

Cycles of Freezing Thawing & Heating	Expansion—Per Cent		Loss in Weight—Per Cent	
	Plain	HR-0	Plain	HR-0
10	0.039	0.034	0.2	0.1
15	.052	.035	.5	.2
25	.096	.047	.9	.2
35	.143	.054	1.4	.3
50	.412	.081	12.3	.5

HR-20

Cycles of Freezing, Thawing & Heating	Expansion—Per Cent					
	Mix A		Mix B		Mix C	
	Plain	HR-20	Plain	HR-20	Plain	HR-20
15	0.020	0.022	0.025	0.030	0.026	0.026
25	.028	.033	.034	.040	.045	.035
50	.045	.055	.047	.056	.074	.054
75	.060	.081	.074	.079	.127	.072
110	.108	.113	.113	.115	.235	.101
Loss in weight after 150 cycles—per cent						
	2	1	8	3	32	7

Note—Each value represents the average of 3 tests made on 3 x 6″ cylinders; see Table No. 3 for strength and mix data.

Specimens of HR-0 group cured 7 days.

Specimens of HR-20 group cured 28 days.

TABLE C—EFFECT OF HR-0 AND HR-20 ON DRYING SHRINKAGE OF CONCRETE

HR-0 (Mix D)

Period of Drying—Days	Drying Shrinkage—Per Cent	
	Plain	HR-0
7	0.019	0.018
28	.047	.046
56	.059	.057
100	.065	.062

HR-20

Period of Drying—Days	Drying Shrinkage—Per Cent					
	Mix A		Mix B		Mix C	
	Plain	HR-20	Plain	HR-20	Plain	HR-20
7	0.022	0.022	0.021	0.023	0.012	0.016
28	.033	.036	.034	.039	.029	.035
60	.038	.044	.042	.047	.040	.047
150	.051	.059	.057	.062	.057	.065

NOTE—Each value represents the average of 2 tests made on 3 x 6" cylinders. See Table 3 for strength and mix data.
Specimens of HR-0 group cured 7 days.
Specimens of HR-20 group cured 28 days.

TABLE D—CHEMICAL ANALYSIS, CALCULATED COMPOUND COMPOSITION, FINENESS AND SPECIFIC GRAVITY OF CEMENTS

Cement	Oxide Composition—Per Cent						Loss on Ignition	Insol. Residue
	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3		
Standard L	21.70	2.20	5.80	64.40	3.00	1.50	0.90	....
Standard U	22.59	4.13	4.99	64.54	1.31	1.29	0.67	....
Standard H			No Data					
Standard L.M.	20.76	2.55	6.07	63.83	3.02	1.71	0.95	0.11
Standard (77)	20.85	2.98	5.73	63.55	2.18	1.97	1.99	0.20
Standard (91)	19.82	2.48	7.07	63.15	3.12	1.91	1.27	0.26
High Early	21.04	3.26	4.95	65.10	1.20	2.26	1.37	0.08
Modified	23.92	1.85	3.49	64.17	2.57	1.65	1.56	0.07
Modified S	21.90	5.47	5.47	61.82	2.36	1.78	0.97	0.37
Sulphate Resisting	24.36	2.42	3.18	64.86	....	1.48	1.09	0.20
Low Heat			No Data					

Cement	Calculated Compound Composition* Per Cent						Specific Surface Area Sq.Cm/g.	Specific Gravity
	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO		
Standard L	46.0	27.0	12.0	7.0	2.5	....	1870	3.148
Standard U	47.0	29.0	6.5	13.0	2.2	....	1770	3.154
Standard H			No Data					
Standard L.M.	47.9	23.4	11.8	7.8	2.9	1.19	1865	3.143
Standard (77)	44.2	26.5	10.1	9.1	3.4	1.88	1870	3.134
Standard (91)	47.5	21.0	14.5	7.5	3.3	0.58	1725	3.141
High Early	57.6	16.9	7.8	9.9	3.9	0.77	2475	3.130
Modified	42.5	36.5	6.1	5.6	2.8	1.49	2040	3.130
Modified S	35.4	36.1	5.2	16.6	....	....	....	....
Sulphate Resisting	49.1	32.8	3.5	8.9	2.5	0.63	2125	3.176
Low Heat			No Data					

*C_3S—Tricalcium Silicate
C_2S—Dicalcium Silicate
C_3A—Tricalcium Aluminate
C_4AF—Tetra Calcium Alumino-Ferrite

TABLE E—CHEMICAL ANALYSIS OF FLY ASH

	Per Cent
SiO_2	44.05
Fe_2O_3	15.44
FeO	3.10
Al_2O_3	21.20
CaO	8.45
MgO	1.03
SO_3	2.30
Loss on Ignition	1.65
Insol. Residue	77.40

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June 1941.*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *NOVEMBER, 1940*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

How to dustproof a concrete floor? (37-64)

Q—Some industrial conditions demand the elimination of dust. If a concrete floor "dusts" what is to be done about it?

*A**—We have dustproofed large areas of industrial floors with varnish. This is somewhat more expensive than burnishing, but it achieves a greater degree of dustproofing for more exacting requirements. We have tested a number of varnishes to determine their relative ability to resist wear.

Some of the heavy brittle varnishes wear longer than lighter-bodied ones, but re-applications do not look so well because of the thickness of the remaining film. Also they are very difficult to remove. The

*By a member of the Institute with much practical experience with this problem but who wishes his name withheld.

lighter varnishes do not last as long, but several coats may be applied before any are removed, and the removal is not so difficult. We prefer the lighter ones.

Our recommendation for dustproofing is as follows:

1. The floor must be free from oil or grease spots.

2. If grease is removed with solvents, the solvents must be washed off using trisodium phosphate or other mildy alkaline detergent. Ordinarily this may be omitted, unless the floor is an old one and is very dirty. After this washing the floor must be allowed to dry.

3. A solution of zinc sulphate may be mopped onto the dry floor. If the floor is new this is fairly necessary, because the zinc sulphate neutralizes free lime in the concrete. Whether the floor is new or not, this serves also to fill pores in the finish. Acting as a filler in this way, the zinc sulphate reduces the cost of the job; it is less expensive than varnish and is very quickly mixed and applied. The solution should be mixed as follows:

1 gallon of warm water
1 pound of zinc sulphate crystals
½ pint of sulphuric acid, 4% solution by volume

It is necessary to let this stand for several days, or until a white powder forms on the surface; then this powder should be brushed off dry. Because of the delay this is not warranted for a small job. A prime coat of varnish would take the place of it. As an alternative to the zinc sulphate wash, a new floor could be washed with a weak solution of muriatic acid.

4. The varnish itself is applied with a stiff floor brush, being brushed out as thinly as possible. The material is poured out into a shallow pan so that only the tips of the bristles can be dipped into it. We have been able to cover 1000 sq. ft. per gal. for the first, and 1500 sq. ft. per gal. for the second coat. One coat should suffice for dustproofing alone, but the second will likely be desirable for a better appearance.

How to prevent corrosion of lead in buildings? (37-65)

Q—More than a year ago a problem arose in connection with the failure of lead-covered cables buried in a concrete floor between a dam and the upstream wall of a power house (September 1939 JOURNAL 36-46). An Institute member calls attention to studies of the corrosion of lead by the British Department of Scientific and Industrial Research as reported in Building Research Technical Paper No. 8, "The Corrosion of Lead in Buildings," and recommendations for "The Prevention of Corrosion of Lead in Buildings"—Bulletin No. 6. The following is digested from that portion of the Bulletin most applicable to the field of concrete:

A—Towards strong hydrochloric and sulphuric acids lead shows a high degree of resistance and is used for lining chambers used in the manufacture of sulphuric acids. On the other hand it is rapidly attacked by dilute organic acids.

Oxygen in the presence of moisture oxidizes lead quite readily and when very soft water is carried by lead pipes the metal may be dissolved and the water become injurious to health. Ordinary hard waters do not dissolve lead since they contain carbon dioxide and this forms a protective film of lead carbonate around the inside of the pipe. Soft waters from peaty catchment areas contain humic acids and are especially corrosive of lead pipes.

Lead freely exposed to the atmosphere is very durable and is valuable for roofs, gutters, etc. It is not oxidized, except superficially, because constituents of the atmosphere, carbonic acid and, probably also sulphuric acid form protective films which prevent further oxidization.

Although known that mortar is capable of corroding lead, the fact is frequently overlooked, with distressing results, as in a corroded lead damp course sent to the Building Research Station for examination. The damp course had been laid in a 9-in. brick parapet wall, there being two courses of bricks bedded in cement above, surmounted by a Portland stone coping. When the wall was taken down after 15 years the lead was found to be partially destroyed. The 5 lb. lead had been converted on the upper side to a brittle mass of lead oxide, leaving in places only a paper-thickness of metal. A comparison of this with several other instances indicates that where lead is bedded in cement mortar corrosion may penetrate the sheet at the rate of about 1/200th in. per year.

Investigation shows the effect is due to the combined operation of moisture, oxygen, and lime derived either from lime mortar or from portland cement mortar. When exposed to the atmosphere lead quickly develops a protective film of lead carbonate; but this does not necessarily occur when lead is placed in contact with cement, as any carbon dioxide in solution is removed by interaction with the cement or lime. Of course, with the passage of time, the action of atmospheric carbon dioxide converts the lime in the mortar to calcium carbonate and the mortar is then innocuous.

When lead damp courses are inserted in thick masonry or brick walls, and especially if portland cement mortar, which carbonates more slowly than lime mortar, is used, some protection of the lead is desirable, since in the heart of the wall the lead may be exposed to lime long enough to allow complete destruction to occur before the

process of carbonation, proceeding inwards from the surface of the wall, renders the mortar inert. A suitable grade of bitumen will afford this protection, or, alternatively, a compound damp course, composed of layers of bitumen-impregnated felt and lead sheet may be used. It should be unnecessary to point out that the use of a composite damp course in which a cover of bitumen is an excuse for reducing the lead to something approaching foil thickness is to be strongly deprecated. In passing lead pipes through walls, where it is desired to make good with cement, the pipe may be protected by a layer of bitumen felt. Another method of dealing with pipes is to pack loosely with *old* lime mortar and then to make up the ends of the hole with a coating of cement mortar.

Tests have shown that aluminous cement, owing to its low content of free lime, is not so harmful to lead as portland cement and lime. The substitution of this cement for portland cement in placing lead damp courses, etc., would be an advantage.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Concrete lace curtains for walls

Engineering News-Record, Vol. 124, No. 23, June 6, 1940, pp. 73-74. Reviewed by S. J. CHAMBERLIN

Thin architectural concrete slabs, hung from the floors, were used to form an openwork curtain around a three-story parking structure. Prepared in the studios of John J. Earley, the slabs provided an economical and attractive exterior.

Preparing the aggregate for Shasta concrete

Engineering News-Record, Vol. 125, No. 3, July 18, 1940, pp. 64-65. Reviewed by S. J. CHAMBERLIN

Flow sheet for the preparation of 10,000,000 tons of sand and gravel for the concrete of Shasta Dam. Dug from the river bank by a walking dragline and delivered to a hopper on skids at the end of a pendulum conveyor, the material is crushed, screened and washed. Four sizes of rock and three grades of sand are delivered to a 9.6 mile belt conveyor which transports the aggregate to the dam.

Model tests, bridge pier supported on long steel piles

THOMAS F. COMBER and JOHN M. COAN. *Proc. Am. Soc. C. E.*, Vol. 66, No. 6 (June 1940), p. 1033-1052
Reviewed by H. J. GILKEY

Tests on models of long tubular steel piles presumably supporting a deep water bridge pier. Restraining action of foundation material was neglected. From the measured strains and pier deflections corresponding values for prototype can be determined. The five conditions of test included: north-south ice, wind load, direct load, combination, and east-west ice.

Flotation as applied to modern cement manufacture

G. K. ENGLEHART, *Industrial and Engineering Chemistry*, Vol. 32, No. 5, May 1940, pp. 645-651.
Reviewed by ROY N. YOUNG

This article is one of several which deal with various methods of separation and which are presented in the same issue. The principles involved in the flotation of cement raw materials are discussed and practical applications of them in cement plants are described. The process widens the range in chemical composition of suitable limestones for the manufacture of cement and facilitates the control of the chemical compositions required for the several types of portland cement.

Belt takes aggregate to Shasta

Engineering News-Record, Vol. 125, No. 3, July 18, 1940, pp. 66-68. ENG. NEWS-REC. ABSTRACT

Most economical transportation on a 9.6-mile haul for 10,000,000 tons of concrete materials to be delivered to Shasta Dam was found to be by 36-in. belt conveyor system consisting of 26 flights 850 to 3240 ft. long. Lengths vary inversely with grades which range up to 25 per cent to reach a high point 960 ft. above the loading level. Each flight is designed for a 200-h. p. motor except on the four downhill flights where regenerative braking is used. Two-way radio patrol cars facilitate close supervision of operation.

Concrete formula based on 6000 tests

N. M. FINKBINER and G. W. HARRA, *Engineering News-Record*, Vol. 125, No. 11, Sept. 12, 1940, pp. 101-102. Reviewed by S. J. CHAMBERLIN

Based on the results from 6000 cylinders the Oregon highway department has developed the empirical formula for obtaining the 28-day strengths from the 7-day strengths of concrete as $S_{28} = 1.51\ S_7 + 49$. The cylinders were cast from four different cements (tabular data given) and a variety of aggregates, and are job cylinders, cured and tested in the laboratory. With a sufficient number of 7-day tests the authors state that the 28-day strength may be predicted within 200 p.s.i. (The significance of the constant 49 is not evident to the reviewer.)

How Friant Dam is being built

Engineering News-Record, Vol. 125, No. 5, Aug. 1, 1940, pp. 40-44. Reviewed by S. J. CHAMBERLIN

The aggregate plant two miles from the dam will produce sand of four sizes, all combined in the stockpile to give a blend that meets requirements. Gravel will be classified into four sizes. Conveyed by belting of a new type the aggregate will be delivered to a loading hopper which has discharge gates arranged to load a 7-car train at one time. From the four 4-cu. yd. mixers the concrete will be delivered by train over a trestle to fast hammerhead cranes and revolving derricks. Ice machines will cool the mixing water, and a 20 per cent pumicite admixture will be used with the low-heat cement.

Comparison of mix designs

N. M. FINKBINER, G. W. HARRA and O. A. WHITE, *Engineering News-Record*, Vol. 124, No. 25, June 20, 1940, pp. 63-64. Reviewed by S. J. CHAMBERLIN

Comparisons were made by the Oregon State Highway Commission on the aggregate void, maximum density, and fineness factor methods of design for mixes of low cement content. The results indicated: that the aggregate void method is laborious and time-consuming because of the difficulty of obtaining the workability factors and the number of void tests necessary; that the quality of the cement has a marked effect on strength; that the maximum density grading produces no additional strength in itself, but permits the use of a lower water-cement ratio due to increased workability with resulting strength increase.

Traveler speeds arch viaduct forms

K. A. SHEPPARD, *Engineering News-Record*, Vol. 125, No. 11, Sept. 12, 1940, pp. 78-80 Reviewed by S. J. CHAMBERLIN

The new approach to the George Washington Bridge consists of concrete Roman arches on steep grades and sharp curves. The formwork was further complicated because of the "board mark texture" required for the concrete. Shaft forms were designed for the 15-ft. lifts without ties by using I-beam wales. A track, supported

by vertical timbers tied around finished piers, carried travelers for supporting the deck and beam forms and the concrete. Wheels were removed from the travelers and the load carried by sections of steel beams except for traveling. The travelers were decked solid to serve as a work platform for erecting, supporting, stripping, cleaning and moving of forms.

Colloidal carbon as a grinding aid in portland cement manufacture

C. W. Sweitzer and A. E. Craig, (Mellon Institute of Industrial Research), *Industrial and Engineering Chemistry*, Vol. 32, No. 6, June 1940, pp. 751-756. Reviewed by Roy N. Young

Each of four lots of clinker from different plants was ground with gypsum and colloidal carbon in a batch ball mill, 18 inches by 18 inches. With constant grinding time for the different batches, the fineness of the cement generally increased with increase in carbon content, but the most effective amounts were from 0.1 per cent to 1.0 per cent. Carbon dosage of 0.32 per cent increased the fineness 30.0 per cent and when the cements were ground to constant fineness the same dosage decreased the grinding time 28.0 per cent. With the exception of color, the properties of cement were not changed appreciably by amounts of carbon up to 1.0 per cent.

Heat of hydration and the choice of cement intended for the construction of dams

LeGenie Civil, Vol. CXVI, No. 18, May 4, 1940, p. 304. Reviewed by R. L. Bertin

The heat liberated by concrete in the process of setting is of little importance in ordinary construction but, in large masses, it has been established that the temperature of the concrete may reach as high as 125° F.

In a paper written by Mr. Stucky, Professor at the Engineering School of Lausanne and Mr. Derron, and published in the October 21, 1939 issue of *The Bulletin Technique de la Suisse Romande*, the subject of concrete requirements for dams is handled in great detail, particularly with reference to the thermal properties of concrete made with different cements. Many tests were made, the results of which are given and many practical conclusions derived therefrom.

1940-1941 Compilation of A. S. T. M. standards on cement

Compilation, Am. Soc. for Testing Materials, 107 p., heavy paper cover, $1.00.

All of the specifications and test methods issued by the American Society for Testing Materials covering cement are included in this latest compilation. Each item is given in its latest form. Various changes made in the standards during the year are incorporated, also the new tentative specifications covering five types of portland cement (C 150 – 40 T), and the newly standardized test for autoclave expansion (C 151 – 40 T).

Other specifications cover portland (C 9 – 38), high-early-strength portland, natural, and masonry cements, and sieves for testing purposes. Testing procedures covered include chemical analysis, sampling and physical testing (including a Manual of Cement Testing and Selected References on Portland Cement). Other standards pertain to the compressive strength of portland-cement mortars, and the fineness of portland cement by means of the turbidimeter.

Copies of the 107-page publication can be obtained from A. S. T. M. Headquarters, 260 S. Broad St., Philadelphia, Pa., at $1.00 per copy in heavy paper cover.

Measuring average particle diameter of powders

ERNEST L. GOODEN and CHARLES M. SMITH, *Industrial and Engineering Chemistry*, Vol. 12, No. 8, Aug. 1940, pp. 479-482. AUTHOR'S ABSTRACT

A self-calculating apparatus for direct determination of surface-weighted average particle diameter of powders has been designed and subjected to practical tests. It consists of a device for determining the average or effective particle diameter of a powder as deduced through Carman's specific surface equation from its permeability to air when the sample is formed into a compact bed. The operation of the device is remarkably convenient, simple, and economical of both time and material. The automatic-calculating feature reduces to a minimum the work of a determination and the chances of making mistakes. In precision and accuracy the method employing this apparatus compares favorably with the usual methods of granulometric analysis for powders.

Manufacture of high silica cements in Russia

LeGenie Civil, Vol. CXVI, No. 14, April 6, 1940, pp. 235-236 Reviewed by R. L. BERTIN

The gradual failure of concrete made with ordinary portland cement where exposed to waters containing certain salts and particularly sulphates led the Cement Institute of Leningrad to study the conditions after ten years. A special cement, highly resistant to the action of sulphate waters was developed.

Briefly, the method consists of adding to the clinker of ordinary portland cement a given quantity of natural materials containing active silica which is previously washed and dried and the mixture ground very fine by the ordinary process.

It is claimed that this cement is economical since the added material is very cheap and does not have to be burned.

The use of this cement has grown steadily in Russia, where in 1938 1,143,000 tons were used or about 16 per cent of the total production of portland cement.

Thermal coefficient of expansion of portland cement

S. L. MEYERS, *Industrial and Engineering Chemistry*, Vol. 32, No. 8, August 1940, pp. 1107-1112. Reviewed by ROY N. YOUNG

Thermal coefficient of concrete was found to vary over a wide range. It was affected by type of aggregate, proportion of ingredients, age of concrete and storage conditions. The quantity of cement gel present influences thermal coefficient of expansion, and the fact that gel content is affected by numerous conditions explains some of the changes in thermal coefficient that may take place in a given concrete. The individual major cement compounds have different thermal coefficients of expansion and are affected differently by various storage conditions. Also, volume change tests show that these compounds are affected differently by humidity conditions in the order: tricalcium silicate, tetracalcium aluminoferrite, dicalcium silicate, tricalcium aluminate. The last is affected very little if at all.

Oxides extracted in Merriman's test of portland cement

E. P. FLINT and P. H. BATES, Division of Clay and Silicate Products, National Bureau of Standards. *Rock Products*, Vol. 43, No. 10, October 1940, pp. 46-47. Reviewed by ROY N. YOUNG

The Merriman sugar test for portland cement is claimed by some to provide an index of thoroughness in the burning of the cement clinker. This test is based on the titration of substances extracted from the cement by a sugar solution. The authors have made a study of the amounts of the various oxides extracted by the

sugar solution using commercial cements and commercial clinkers ground with and without the addition of gypsum. With a few exceptions, all of the cement constituents go partly into solution. Generally the amounts of the oxides dissolved are in the order from high to low—CaO, Al_2O_3, SiO_2, Fe_2O_3, (Na_2O and Kn_2O), MgO and SO_3. Clinker ground with gypsum gave much lower titration values than the clinker ground without gypsum. From these results it follows that the sugar test is not a reliable index of the degree of clinker burning.

Proceedings, Highway Research Board, Vol. 19

Proceedings, Highway Research Board, 578 pages, $1.75.

Volume 19, *Proceedings*, Highway Research Board is divided into sections on Finance, Economics, Design, Materials and Construction, Maintenance, Traffic and Safety, Soils and Soil Stabilization.

Contents of special interest in the field of concrete include:

Bond Tests on Rusted Bars, H. J. Gilkey, S. J. Chamberlain, R. W. Beal

Continuous Reinforcement in Concrete Pavements, E. C. Sutherland, S. W. Benham.

Stresses in Concrete Runways of Airports, H. M. Westergaard.

Durability Investigations, A. T. Goldbeck.

Frost Action, A. F. Gill, J. F. J. Thomas.

Curing Concrete Pavements, Bert Myers.

Louisiana tries low heat cement

H. L. Lehman and E. L. Erickson, *Engineering News-Record*, Vol. 125, No. 11, Sept. 12, 1940, pp. 97-98. Reviewed by S. J. Chamberlin

Reports the use of a low-heat cement in the construction of the Baton Rouge bridge to reduce cracking. Rejecting the use of less cement content or of coarser grinding, a cement was specified and used that differs from that for standard portland only in that tricalcium aluminate is limited to 7 per cent and the ratio of the percentage or iron oxide to aluminum oxide to 1.50. Check tests with standard cements indicated that the low-heat cement compared favorably in strength with them and with less tendency for cracking. Results secured in the piers were verified on a roadway paving contract. Besides the reduction of cracks, the slower setting time was beneficial in the construction of lip curbs and hand finishing.

Particle size distribution of portland cement and what it means

F. O. Anderegg, Ph.D., *Rock Products*, Sept. 1940, pp. 52-54. Reviewed by Roy N. Young

Two commercially ground portland cements and five cements of designed particle size gradings were studied with respect to the effect of these gradations on the properties of cement. Both the commercially ground cements had "humps" in their grading curves. Tests showed that the cement having the lesser "hump" had the more desirable properties. The cements of designed grading were produced by obtaining fractions of a cement with air elutriators, using air free of H_2O and CO_2 and then recombining fractions according to design. Test results of four such cements ranging in gradation ratios from 1.00 to 1.18 are given. The cement having gradation ratio 1.00 (equal amounts of fractions, 2.5 to 5, 5 to 10, 10 to 20, 20 to 40, 40 to 80 microns) gave highest strength, densest mortar and lowest volume

change. Replacement of coarse particles (larger than 20 microns) in the designed cement with silica grains of the same size grading led to the conclusion that the coarse cement particles made better "aggregate" than the silica grains. The present trend toward finer grinding of the coarse particles of cement should result in all-around better performance per unit cost.

The hardening of aluminous cement at high and low temperatures

STEVEN GOTTLIEB. *Cement & Lime Manufacture*, Vol. 13, No. 4, April 1940, pp. 55-60.
Reviewed by J. C. PEARSON

The chief interest in this article is in the author's experience with aluminous cement concrete at low temperatures, which, as he points out, may be of use in the construction of concrete fortifications and other works in winter, as actually happened in the Russo-Finnish war. In one case cited, a floor 54 in. thick was cast at a temperature of 0° F., and the temperature did not fall below 32° F. at 4 in. below the surface, nor below 40° F. at a depth of 24 in. When forms were removed at 30 hours, the concrete resounded well when struck with a hammer. Tests made on cubes cut from the upper part of a similar floor exposed to temperatures of 10° to 20° F. showed a compressive strength at 40 hours of 2560 p.s.i.; from the lower part 5340 p.s.i. At temperatures of about 30° F. tests were carried out with concrete containing 500 lb. of cement per cu. yd. and 1-in. aggregate, water-cement ratio .70. Concrete was mixed for 2, 4, and 15 minutes, and 8 in. cubes were cast from each mix. Results as follows:

Mixing Time, Minutes	Compr. Strength, p.s.i.		
	24 Hr.	48 Hr.	7 Days
2	0	1190	4720
4	2060	4680	5460
15	3420	3500	4000

During hardening the temperature varied from 17° F. to 22° F. during the first 48 hours, and from 17° to 34° F. between 48 hours and 7 days. The surface layers of the first two were frozen, but the third was O.K. Tests indicate that long mixing is favorable to early hardening, but at some sacrifice in later strength.

The practical values of the modulus of elasticity of concrete

LeGenie Civil, Vol. CXVI, No. 15, Apr. 13, 1940, pp. 252-253. Reviewed by R. L. BERTIN

This article is a review of a paper by Professor Bolomey, contained in *Bulletin Technique de la Suisse Romande* of Aug. 26 and September 9, 1939 which deals with the modulus of elasticity of concrete.

Professor Bolomey reasoned that while the initial modulus is dependent upon the proportions, the gradation of the aggregate, the quantity of Water and the age, all factors on which the compressive resistance also depends, the formulas expressing the modulus in terms of strength did not yield satisfactory results because they do not take into account the compaction of the concrete.

He states that the modulus of concrete is controlled by the deformation of the pastes on the one hand and that of the aggregate on the other, the former varying with the stress and the latter being nearly constant. With this in mind, a series of tests on concretes, consisting of a constant cement paste and aggregates of different

gradation, was undertaken from which Professor Bolomey derived the following formula:

$$E = (C + S)^2 \frac{700\,R}{100 + R}$$

in which E = Modulus of Elasticity in tons per sq. cm.
C = Ratio of Volume of Cement to that of Concrete.
S = Ratio of Volume of Aggregate to that of Concrete.
R = Compressive Resistance in kgs. per sq. cm. (determined on cubes)

which is claimed to agree with the effective modulus within 6 per cent.

Several observations and conclusions, dealing with the variation in the value of "*E*" resulting from segregation: age, intensity of load, and permanent deformation, are treated at length.

Current research on structural design of concrete pavements

A. A. Anderson, Portland Cement Association. Presented Highway Research Board.
Highway Research Abstracts

Current concrete pavement practice, both in design and construction, is based to a considerable extent on past research which developed the fundamental data necessary to progress. As a measure of the extent of this past research, a summary of 1385 investigations listed in a compilation of Highway Research by the Committee on Research Activities of the A. A. S. H. O. and the Highway Research Board revealed that 659 were directly related to concrete and 436 more were as applicable to concrete as to other surface types. While all these projects do not have a direct bearing on design, it is difficult to conceive of a research project that will not affect design problems, at least indirectly. However, this review has been limited to current research bearing directly on such design problems.

This survey covers 41 States, of which 16 reported no work on concrete research, and the remaining 25 reported from 1 to 8 investigations, each. Current research in the Public Roads Administration was not included because it is regularly reported in "Public Roads". University and college work has not been included. A table in the original paper summarizes the results of the survey. Indicated are the classification of investigations and the number of each being carried on. It should be noted that these are investigations now under way on which final conclusions have not yet been reached, and while it is not a complete list of such work, it does indicate the wide extent of activity in this field.

A study of the research reported as active shows that there is a duplication of effort in some fields and that others are not adequately covered. A correction of this situation would result in better progress and make results of research more quickly available. It is suggested that this could be accomplished through a cooperative arrangement between the Highway Research Board and the A. A. S. H. O. by means of which a committee would determine research needs and outline activities necessary to complete and supplement current investigations. It would seek the cooperation of all research agencies in a "share-the-work" movement that would effect the best progress. Such a program will be no small task, but the inestimable value of the results of such an effort make the undertaking one to be strongly urged and recommended.

Concrete test road in Ohio

R. R. LITEHISER and C. W. ALLEN, Bureau of Tests, Ohio Department of Highways, Sept. 1940. HIGHWAY RESEARCH ABSTRACTS

Early in July, the Ohio Department of Highways started the construction of a concrete test road on U. S. 40 west of Columbus. A principal feature in the 49 test sections will be the use of cements and combinations of different cements manufactured with and without grinding aids. Other features will be sections with an admixture added to the concrete at the time of the mixing and sections with the surface hardened with a mixture of iron, sand and portland cement. In order that a direct comparison can be made between these sections and the regular type of concrete pavements, control sections made with the usual materials are included.

Three brands of natural cement will be used, in several combinations with and without grinding aids. These grinding aids are beef tallow, vinsol resin, and petroleum distillate. The amounts to be used are very small, being less than 1/20 of one per cent of the total weight of the cement. One brand of portland cement will be used, which will be manufactured without the grinding aid, with beef tallow, and with vinsol resin. The percentage of grinding aid will be even smaller than in the natural cement. A portland-puzzolan cement manufactured from a blend of portland cement clinker and puzzolan will also be used. The puzzolan used in this cement is similar to that used by the Romans twenty centuries ago. A waterproofed cement consisting of a standard portland cement interground with a calcium stearate powder completes the varieties of cements used on the project.

The effect of the type of finishing will also be studied, as one-half of the project will be "struck-off" and finished with the regular type of finishing machine while the other half will be "struck-off", compacted and finished with a special vibratory machine. This machine is equipped with an 18 in. front screed on which are mounted 4 electric motors and vibratory units operating at a frequency of 3600 pulsations per min. Previous tests made by the Ohio Department of Highways indicate that concrete pavements finished with this type of machine should show a substantial increase in strength over the usual methods of finishing the pavements.

*Stringconcrete**

AXEL ERIKSSON, *Teknisk Tidskrift*, No. 30, July 27, 1940. Reviewed by BENGT. FRIBERG

"Stringcrete" is a new name in the concrete art to describe a prestressed concrete material in which the reinforcement consists of small high tension wires. The steel wires are stressed to 150,000 to 200,000 p.s.i. prior to concrete placing, and the end stressing arrangement of the wires released after the concrete has gained some strength. The bond between the concrete and the steel, as represented by adhesion and friction, increases in relation to the cross-sectional area of the wire with decreasing wire diameter. The initial compression in the concrete and the corresponding decrease in the initial tension of the steel serve to increase the surface friction between the two materials. For wire diameters of less than .2 in., end anchorage is stated to be unnecessary for the high stresses employed. Only small sizes of cold drawn wire can be obtained in the very high strength of 300,000 to 400,000 p.s.i., and the practical wire diameters have been from .06 in. to .08 in. diameter. The particular type of pre-stressed wire reinforced concrete has been devised in Germany by Evald Hoyer, and Swedish manufacturing license has been obtained by an organization headed by the author.

*The name "Stringcrete" is proposed by the abstractor to differentiate this type of reinforced concrete material.

Interesting photographs of tests and manufacturing are included, with data on flexure tests of 2 in. and 2¾ in. thick slabs and beams of 9½ in. depth. The tests have indicated that at the ultimate load the load stress is not added to the initial stress in the reinforcing wires. Tension cracks in the concrete were observed at approximately one-half of the ultimate load. For the ultimate load the conventional concrete stress formulas gave close results for the materials used. In 4¾ in. x 9½ in. concrete beams, reinforced with .15 per cent steel in the upper edge and .39 per cent steel in the lower edge, prestressed to 200,000 p.s.i. in the .08 in. diameter wire having a tensile strength of 340,000 p.s.i., tension cracks in the bottom of the beams were first observed under a modulus of tension of 3000 p.s.i., with failure (average of 11 beams) at 6100 p.s.i., figured according to the homogeneous material flexure formula. The same first crack and ultimate stress values governed for both the beams and the slabs, indicating that for practical use Stringcrete computations can be made by the standard bending formula.

A substantial quantity of precast Stringcrete beams have been furnished for reinforcement of public air raid shelters in the City of Stockholm. The weight of Stringcrete beams has been three times that of equivalent steel beams. The material is of greatly smaller cross section than conventional concrete members; for that reason the reinforcing percentages, even considering the very high strength of the reinforcing materials, are relatively great and the reinforcement is placed in two, four, or six layers. For factory manufacture of Stringcrete precast members at a price of $1.25 per cu. ft., it is estimated that a production of 1500 cu. ft. per day would give satisfactory return on invested capital.

Resistance to freezing and thawing

W. H. Wood, Texas State Highway Department, presented National Sand and Gravel Association, St. Louis, Mo., Jan. 17, 1940. Highway Research abstracts

The relative resistance of fine or coarse aggregate alone to alternations of freezing and thawing does not necessarily reflect the relative resistance to alternations of freezing and thawing of comparable concrete made with the respective aggregates. Aggregates showing high resistance to freezing and thawing may or may not produce concrete relatively low in resistance while aggregates showing relatively low resistance to freezing and thawing may or may not produce concrete relatively high in resistance. Much depends on the nature and surface characteristics of the aggregate.

The shape and surface texture of the coarse aggregate affect the durability of the concrete. For the aggregates covered in this investigation it was found that when subjected to repeated cycles of freezing and thawing, limestone gravel and crushed limestone afforded better bond between the mortar and the coarse aggregate and produced more durable concrete than that made from well rounded somewhat slick siliceous gravel, although the latter showed much better resistance to such alternations when tested alone.

With a given set of fine and coarse aggregates the durability of the concrete is affected by the water-cement ratio, greater durability being obtained as the water-cement ratio is lowered. When the relative durabilities of concretes made with several different sets of fine and coarse aggregates are considered, however, the effects of the water-cement ratio may be entirely offset or overshadowed by the types of aggregates employed. It may be found that concrete with a water-cement ratio of 1.2 (9 gal. per sack of cement) and made with one set of aggregates may

be much more durable than concrete with a water-cement ratio of 0.8 (6 gal. per sack and made with another set of aggregates.

With a given set of fine and coarse aggregates the durability of the concrete is affected by the strength of the concrete, greater durability being obtained as the strength is increased. This relationship is true for either compressive or flexural strength. When the relative durabilities of concrete made with several different sets of fine and coarse aggregates are considered, however, the effects of the strength may be entirely offset or overshadowed by the types of aggregates employed. For instance, it may be found that a concrete having a compressive strength of 3000 p.s.i. with one set of aggregates may be much more durable than a concrete having a compressive strength of 5700 p.s.i. but employing another set of aggregates. Also it may be found that a concrete having a modulus of rupture of 560 p.s.i. with one set of aggregates may be much more durable than a concrete having a modulus of rupture of 745 p.s.i. but made with another set of aggregates.

The use of limestone rock "flour" mineral filler to improve the workability of a concrete mix did not affect the durability of the concrete adversely.

It is indicated that decreasing the density of a concrete mix by increasing the total mixing water affects the durability of the concrete adversely even though the water-cement ratio is kept constant by increasing the cement factor.

Compression testing of cores secured at various times from concrete pavement slabs over different parts of Texas has shown that the concrete is continuing to gain in strength after undergoing natural weathering, including freezing and thawing, for periods up to five years even though the water-cement ratios of most of the pavement tested were considerably in excess of 0.8 (6 gal. per sack of cement).

Masonry dams, a symposium

Proc. Am. Soc. C. E., Vol. 66, No. 5, May 1940, p. 811-943. Reviewed by H. J. GILKEY

Basic design assumptions

IVAN E. HOUK and KENNETH B. KEENER (p. 813-828)

Treatment restricted to high masonry dams of single arch, curved gravity, and straight gravity types built on rock foundations and to those assumptions which embody recent new developments. Items considered are: the dam site, the dam, load conditions, structural action, stability factors, and stress conditions. Includes an appendix of 38 references to some of the more significant recent writings on the aspects covered.

Design of arch dams

R. S. LIEURANCE (p. 829-851)

Discusses briefly the design problems involved as a preliminary to the presentation of an appendix, containing a comprehensive series of tables to facilitate the computation of forces, moments, and radial deflections in the design of arch dams. Seven basic load conditions are provided for in the appendix.

Preparation of foundations

CHARLES H. PAUL and JOSEPH JACOBS (p. 852-868)

Except for the discussion of a few "Special Conditions" only rock foundations are considered. Discusses bearing capacity, bond, frictional resistance, uplift pressure, seepage, scour, exploratory core drilling (large cores), rough and final excavation, seams, grouting, drainage, and special abutment treatments. Emphasizes

that each site constitutes a special problem and that sound seasoned judgment in foundation matters is as indispensable as is expert knowledge.

Geological problems of dams

IRVING B. CROSBY (p. 869-890)

Treatment confined to the consideration of geological and foundation problems of masonry dams founded upon rock. Cites illustrations of successful dams and of dams that have failed on different types of rock and geologic structure. Concludes that the essential foundation conditions can be determined in advance and that at most sites safe dams can be built and that dam failures are avoidable. Outlines methods for geologic investigation and discusses the specific problems relating to dam construction on different types of foundation rock.

Concrete control

I. L. TYLER (p. 891-907)

Discusses the special cements developed for use in recent dams, as regards composition, fineness and puzzolen as an admixture. Aggregates are discussed with regard to type, grading and shape of particles. The design of the mixture, its placement, the clean up, curing, cracking, and tests, all receive attention. An appendix with eleven current references is included.

Construction joints

BYRAM W. STEELE (p. 908-943)

Treatment relates to a formed or unformed horizontal, vertical or inclined surface between masses of concrete placed at different times. If cracking is to be avoided or controlled, joints must be placed closer together than was general under past practice. Paper discusses joint spacing, plain vs. keyed joints, slots vs. grouted joints vs. open joints, joint surface treatment, bonding of joint surfaces, grout stops, joint fillers, grouting and height of lift. In a comprehensive appendix, data are supplied on 78 of the world's leading masonry dams each entry being supplemented by additional discussion wherever special factors were present. Supplementary data are also included on special types of cement when used.

What do we know about concrete slabs?

N. M. NEWMARK, *Civil Engineering*, Vol. 10, No. 9, Sept. 1940, pp. 559-562.

Reviewed by J. R. SHANK

This paper is an abridgement of a paper which is on file in the Engineering Societies Library. It is a concise review of analytical considerations and experimental results and is in plain English in the form of general statements without mathematical deductions. As the paper is already an abridgement, no adequate abstract can be contained in small space. This so-called abstract will be in the form of quotations more or less connected.

"Any approach to the design of concrete slabs that rests entirely on empirical considerations or entirely on analytical considerations is bound to be ineffective." "We know, . . ., from tests, that in general the reinforcing steel will work before failure . . ." "We are sure that in certain cases the total moment on a section, or the sum of the moments on several sections, has a particular value determined by statics. We have some reason to believe that if we provide for this total with reasonable arrangement of reinforcing, we shall have probably as good a design as we are likely to attain by other more elaborate analyses."

The author deals with the analytical methods which apply to homogeneous elastic materials, gives a statement of the fundamental assumptions generally used,

and outlines some of the fundamental relations involved and some of the methods used. "Static Controls" are discussed as checks or guides to the analyses. Among these are: "For example, in a square slab uniformly loaded, not fixed or continuous at the edges, the average moment per unit width across the diagonals is 1/24 the total load on the slab."; and " . . ., in a rectangular panel of a simply supported slab, the total moment in the slab and the supporting beams on any section parallel to one pair of supports is equal to the static moment of the loads." "When the load is relatively highly concentrated, the stresses in the vicinity of the load are not accurately given by the ordinary theory of flexure. The stress distribution on a line through the center of the load" is not nearly linear.

Concerning the shearing stresses in reinforced concrete slabs he says: "Since a rectangular slab is generally very shallow compared with a concrete beam having the same span, the shearing stresses in a slab are generally not critical. There is considerable evidence that a rectangular slab of ordinary proportions that is adequately designed for flexure will be adequate in shearing resistance."

"The departures of the properties of reinforced concrete slabs from those assumed in setting up the analysis are in general of such a nature as to permit the theory to be on the safe side."

"We may expect theoretical studies to indicate greater stresses in a reinforced concrete slab than the stresses actually existing at relatively high loads." "Before failure of the structure can take place, the slab and its supporting beams possibly can act so as to apportion the total moment between them in the most effective manner. Failure on any section will not take place in general until nearly all of the steel crossing the section reaches the yield point." "When the deflections of a slab without lateral restraint at the edge becomes nearly equal to the depth or greater than the depth, the direct stresses in the slab become important in contributing to the carrying capacity of the structure. A slab with large deflections acts in some respects as an inverted dome. The bending stresses at the center are decreased but direct tensions are added at the center." "When the edges of the slab are restrained against lateral movement, high tensile stresses can be developed with even small deflections, and a considerably larger reduction in stress at the center of the panel can be effected."

Concerning what has been learned from experiments: "It seems necessary to point out that a comparison of computed and measured deflections of a slab, as a means of verifying the accuracy of the theory, is almost worthless." "Examination of experimental data seems to indicate that the concrete in a slab cracks at about the computed moment required to produce the ultimate tensile strength in the concrete at the top or bottom of the slab." " . . . the actual factor of safety against cracking of a reinforced concrete slab is exactly the same as in a corresponding beam of the same depth and reinforcement." Concerning tests reported by Richart and Kluge: "The measured steel strains are almost without exception less than the theoretical steel strains computed on the basis of no tension in the concrete, but are greater than for full tension in the concrete, except for points near the supports. The steel strains near the load become more nearly equal to the theoretical values for no tension in the concrete as the load becomes large."

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Fire Damage to General Mills Building and Its Repair*

By J. Fruchtbaum†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The repair of the General Mills building at Buffalo, N. Y. after the fire of February 15, 1940 presented many unusual problems—replacing large areas enclosed on all sides by portions of the building remaining intact. This problem was further complicated due to the number of floors involved.

Literature on repairs of a fire damaged job is lacking in specific detail. The effect of heat on the strength of concrete and steel has received little attention and the available information was insufficient to permit a determination of what should or should not be replaced in border line cases.

The engineer in charge of this work conducted the necessary tests even to cutting and measuring carrying capacity of columns involved.

A complete program, based on these data was evolved. Unusual details were designed and detailed for all the complicated cases that a flat slab presents. The repairs were made on schedule. Careful records were kept of the progress and costs. Check tests of loading and deflections on repaired panels were made. All of this information is presented in this paper.

INTRODUCTORY

The rebuilding of the cereal factory in Buffalo, N. Y. (Fig. 1) damaged seriously by fire in February 1940, presented several interesting problems in replacing areas that were enclosed on all sides by portions of the building remaining intact.

THE BUILDING

In general, the structure is nine stories high, 316 ft. 6 in. long, 76 ft. wide. The floors, except for the two bays at each end and a few interior bays, are of flat slab construction, with drop panels and

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†Engineer, Reecon Co., Buffalo, N. Y., in charge of the work described.

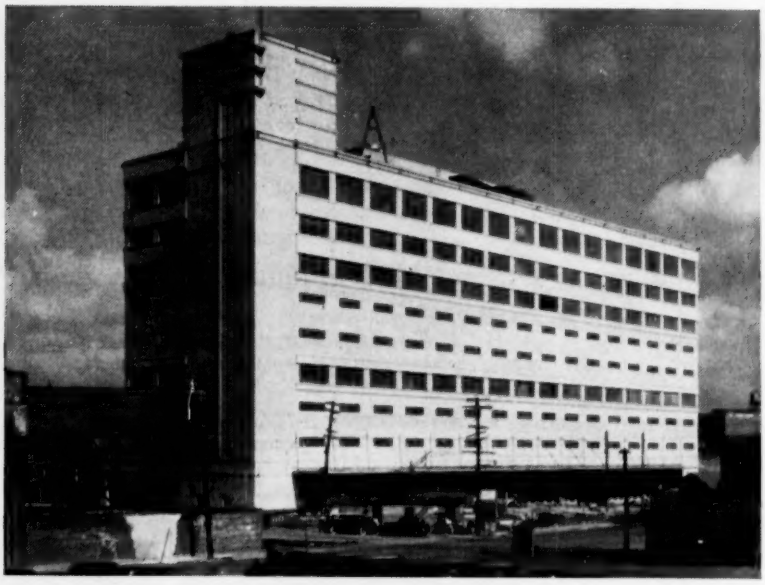


FIG. 1—GENERAL MILLS BUILDING—REINFORCED CONCRETE; DAMAGED BY FIRE AND REPAIRED

column capitals. The rest of the structure is of the beam and girder type. All the columns are spiral reinforced.

The walls are of unglazed facing tile, vertical cell construction; windows of steel sash and glass block. (See plan and section, Fig. 2).

Urgent need for the new space led to occupancy before completion of the sprinkler system and other interior subcontracts.

The building design was prepared by the Cereal Engineering Co., Minneapolis, Minn.; detailing and changes in design necessitated by the addition of bin supports inside the building, changes in column loads, location of new openings, and the relocation of others, were by Reecon Co., Buffalo, N. Y.

THE FIRE

The fire started on the fourth floor about 11:10 a. m. Feb. 15, and spread in a northwesterly direction, due to the method of fighting the fire, and the location of the openings in the floor above. It was smoky, and attained temperatures varying from about 900°F. on the outside borders of the combustible material, to about 1300°F. in the area bounded by columns D-10, D-12, C-14, B-14, B-13, and C-11 (Fig. 2). As the fire continued, it returned in a southeasterly direction toward column row 17. By about 7 p. m. the fire was out on this

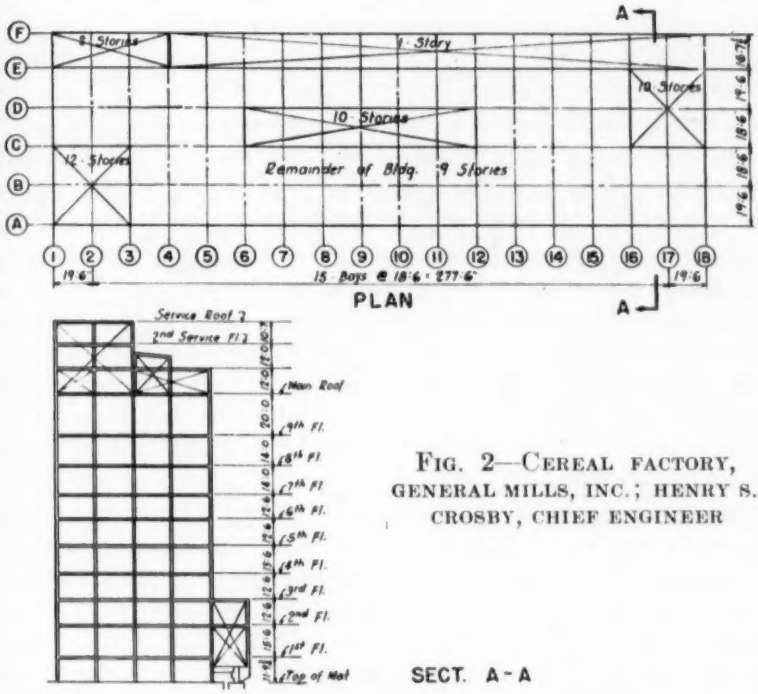


FIG. 2—CEREAL FACTORY, GENERAL MILLS, INC.; HENRY S. CROSBY, CHIEF ENGINEER

floor, but it is doubtful if any one area was subjected to the full heat for more than three or four hours.

The material stored consisted of cardboard cartons containing premiums, (toys made of cardboard and metal) piled to within three feet of the ceiling, in the section affected by the fire.

In about 20 minutes after the outbreak of the fire on the fourth floor it reached the fifth floor primarily through the central spiral chute. It spread westward between column rows D and B, and then east along the north wall. Soon it covered practically the entire floor, but the serious damage was more or less localized in those storage areas, where the fire gained headway. An emergency sprinkler connection was effected which helped enormously where the pipe hangers remained in place. Approximately 17,000,000 gallons of water were fed into the sprinkler system by General Mills pumps.

Sections of the fifth floor slabs were exposed to temperatures of over 1200°F. for probably six hours. The maximum temperatures of over 1700°F. were probably of only two to three hours duration. The hottest portion of the fire was probably in the area bounded by column rows C, B, 7 and 8, as was later shown by the fact that three of these four columns had to have their heads replaced. The fire raged until

11 p. m. but was still smouldering the next morning. There were fused vapor proof globes at columns B-8, C-8, C-9 and D-7. A drooped sprinkler deflector was found on this floor but whether it came from the sixth floor ceiling or this ceiling is impossible to say.

The material stored on this floor consisted of cartons of Kix (puffed corn cereal packed in waxed containers), loaded to a height of 9 ft. 9 in., over practically the entire area between column rows 3 and 16.

In about an hour after starting, the fire spread to the sixth floor and again, primarily, through the central spiral chute. It moved rapidly eastward among the cartons of Kix that were stored almost solid between column rows 9 and 15, and the area bounded by column rows C and E and 3 and 9. (There was no storage in the bay bounded by column rows 7, 8, D and E and there were 6-ft. aisles on the north and south sides). After a good start, the fire spread westward between column rows C and E.

The maximum heat developed near the top of the storage pile along the center line of column row C between rows 9 and 13. Evidence of calcining of concrete was found. Some of the vapor proof globes at columns C-8, C-9 and D-7 flowed down and assumed the shape shown in Fig. 3. Glass jars stored in cereal cartons fused. Yet at the west end of the building a wall panel was scorched but solder did not melt.

In the area bounded by column rows B, D, 10, and 15, a temperature of about 2000°F. was reached and high temperatures were maintained for probably 10 to 12 hours. From that area out, the temperature dropped to about 800°F. at the column rows A, 3, and 17. The temperature seems to have reached high points (about 1400°F.) in bay 5, 6, D, and E, and bay 12, 13, D, and E. The total duration of severe fire on this floor was about 15 to 18 hours. Smouldering continued for two more days.

The fire spread to the seventh floor sometime after 4:00 p. m. through the service shaft, or east spiral chute or both, but assumed severe proportions between 9:00 and 10:00 p. m. It is to be noted that it took at least three hours, and probably over four, for the fire to travel between the sixth and seventh floors. This was due primarily to the central spiral chute stopping at the sixth floor level. The action of the firemen, fighting the fire from the east stair tower prevented the spread through the east chute.

At midnight, only a slight amount of water had been poured on the seventh floor, and no real application of water was made at the center of the building until late next morning. This made for a very severe

FIG. 3—GLASS JARS AND GLOBES FUSED IN FIRE

fire showing a good deal of open flame, especially as the concrete began to spall. The fire spread east over the entire area where Kix were stored (column rows 9 to 16, except for six-foot aisles at north and south walls). The fire lasted about 24 hours and attained the maximum heat in the bay bounded by column rows B and D and 11 and 12, and along the center line of column row D between 8 and 9. The fire reached temperatures of about 1800°F. Evidence of calcining of concrete was found. Opening a door in the west end of the building

leading to the stair tower caused a five-foot flame to leap toward the stairs.

There was no fire above the eighth floor, but as the concrete was spalling away from the underside, the floor began to sag under the load. The temperature on the upper side of the floor slab was such that walking on it was impossible the next morning. The temperature at the ceiling of this story was high enough to open sprinklers over an area of 3500 sq. ft.

Floor bay 11, 12, C, and D was loaded with bags of salt weighing 300 lb. per sq. ft. Saturation increased the load to about 350 lb. per sq. ft. Bays 9, 11, C, and D were loaded with sugar weighing 300 lb. per sq. ft.; bay 9, 10, B, and C, with drums of malt syrup, weighing 200 lb. per sq. ft.

While there was no fire on the ninth floor enough smoke and heat did get through the bins at column rows 9, 10, D and E to open nine sprinklers.

Automatic Sprinklers

At the time of the fire automatic sprinklers of 160°F. rating had been installed on 9 ft. 3 in. by 9 ft. 11 in. spacings in the fifth and stories above. None had been installed in the lower stories, nor had the piping been completed below—conditions which account for the delay in getting water into the sprinkler system. The number of sprinklers opened in each of the upper stories were: 5th story, 292 heads; 6th, 286; 7th, 287; 8th, 38; 9th, 9. These numbers represent practically the total installations in the fifth, sixth and seventh stories.

Photographs were made of the under side of each of the floor slabs affected by the fire and assembled for study. Fig. 4 is one of these details and Fig. 5 and 6 give a further idea of the damage. Fig. 7 and 8 show a crack survey of fifth, sixth, seventh and eighth floors. These show the original construction joints and the areas cut out and replaced.

No material fell through from floor to floor—This, in spite of the fact that when the debris was cleaned up, loose pieces of concrete were resting on exposed bars. We believe this was due to the method used in placing the reinforcing steel. The intermediate bands rested on the main bands in all cases, and the main bands had the additional protection of the drop panel and column capitals.

Aggregates

It is generally agreed, that because of the easier readjustment of the smaller particles, the effect of the fine aggregate on fire resistance

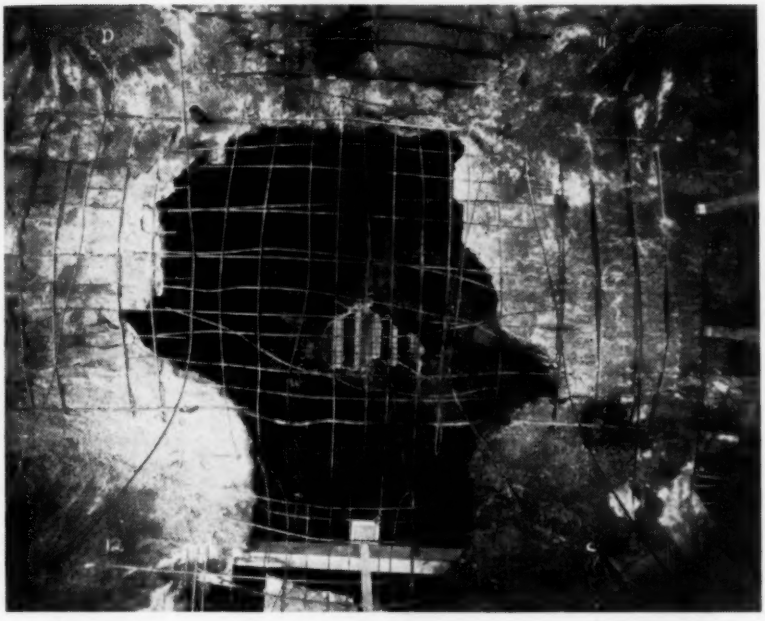


FIG. 4—FIRE DAMAGE TO SEVENTH FLOOR SLAB FROM BELOW—NOTE HOLE IN EIGHTH FLOOR SLAB

is limited. We look therefore to the coarse aggregate for an explanation of the serious spalling. While the composition of glacial deposits must be considered a variable item, there is nevertheless a reasonable agreement between several tests made on aggregates from the same source of supply during the life of the job, to show the following rational analysis: quartz, 55 per cent; feldspar, 30 per cent; calcite and dolomite, 5 per cent; kaolin, 5 per cent; R_2O_3, 5 per cent.

Chert, if any, is grouped with quartz, because of the methods of analysis used, and its occurrence in the local gravels may be considered the exception. An examination by Herbert Insley, Senior Petrographer, National Bureau of Standards showed a sample of Lake Erie gravel as used on this project to consist of: quartzite, 30 per cent; sandstone, 45 per cent; granite, 10 per cent; dolomite and limestone, 5 per cent; chert, 10 per cent.

The number of variables involved in fire effects is such, that exact determinations are not possible. It is known, however, that chert disrupts violently on being heated to 410°F., and the small pebbles resulting from the explosion burst in turn. It is the action of these chert particles that caused considerable damage to the fifth floor slab.

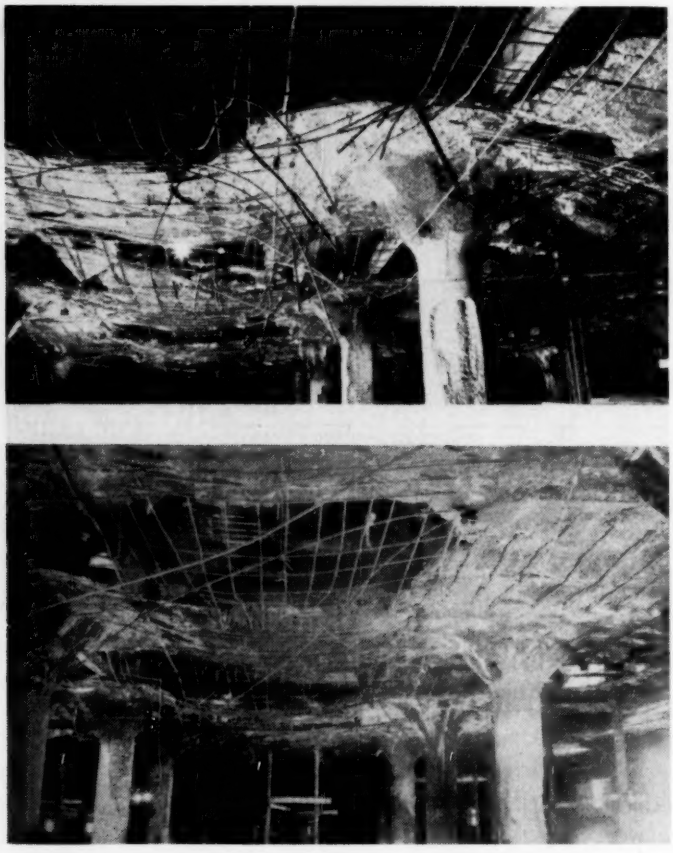


Fig. 5 AND 6—NATURE OF DAMAGE AT ITS WORST—LOOKING UP AT SIXTH FLOOR CEILING

Quartz particles begin to expand rapidly at a temperature of 1063°F. The effect on granite would depend on the coarseness of the grain, the coarser the grain the more serious the damage. The effect on sandstone and quartzite would, I believe, be determined to a large degree by the cementing material. The shape of the particles seems to have little to do with fire resistance.

Limestone is an excellent aggregate from the point of view of fire resistance. The outside calcines and insulates the rest of the member. Streams of water may, of course, wash off the surface, and increase the depth of the damage.

Trap rock and blast furnace slag occupy an intermediate position between quartz and limestone, but in long continued fires will make a better showing than even limestone.

An excellent study and list of references on the fire resistance of aggregates appeared as the Report of A. C. I. Committee E-4, Nolan D. Mitchell, Chairman.* The "Red Books" of the British Fire Prevention Committee, published by the British Department of Scientific and Industrial Research, and a paper by S. H. Ingberg in the 1929 A. S. T. M. *Proceedings* are especially recommended.

Spalling

Layer after layer of concrete dropped off as the fire continued. In the columns, and in many sections of the floors, the reinforcing held the concrete, though loosely, even after it spalled, and prevented the heat from getting deeper into the structural members.

Corners of drop panels sheared off even in locations five to ten feet away. About 40 column capitals poured originally to the under sides of drop panels, were cracked and almost completely sheared off along the surface of the spiral. This separation was not found in any case where the column capital was monolithic with the slab, although cracks, especially along pipes running through the capitals, were general in the fire area. No spalling inside the spiral was found but a loss in compressive strength of about 50 per cent was shown by several cores taken from the center of the columns.

The maximum stresses will, of course, occur at the bottom of the ceiling slab (area nearest to source of heat). Spalling, however, will take place on a section one to two inches back, along some surface where the ratio of stress to resistance would be maximum. Obviously, at the under side of the slab there is room for expansion in one direction (downward), and failure there would be unusual.

Damage from Entrapped Moisture

In several cases, sections two to four square feet in area and three or four inches thick popped. Some of the "pops" were violent. At the sixth and seventh floors $\frac{3}{8}$ in. stirrups were bent as much as two inches out of plumb. In addition to the disruptive effects of the temperature differences through the structural slab and the explosive breaking up of aggregates the moisture remaining in the slab may have added to the violence. A good deal of moisture remained in the 5 or 6 month old slabs. It is possible that where this moisture was trapped in the slab it became superheated above the boiling point, and as internal cracks developed a part of the water flashed into steam to add to the force of the explosion. While the porosity of even dense concrete would make it difficult to build up such pressure, the explosion, however, seemed too violent, and the masses affected too

**Proceedings*, Amer. Concrete Inst., Vol. 21, 1925.

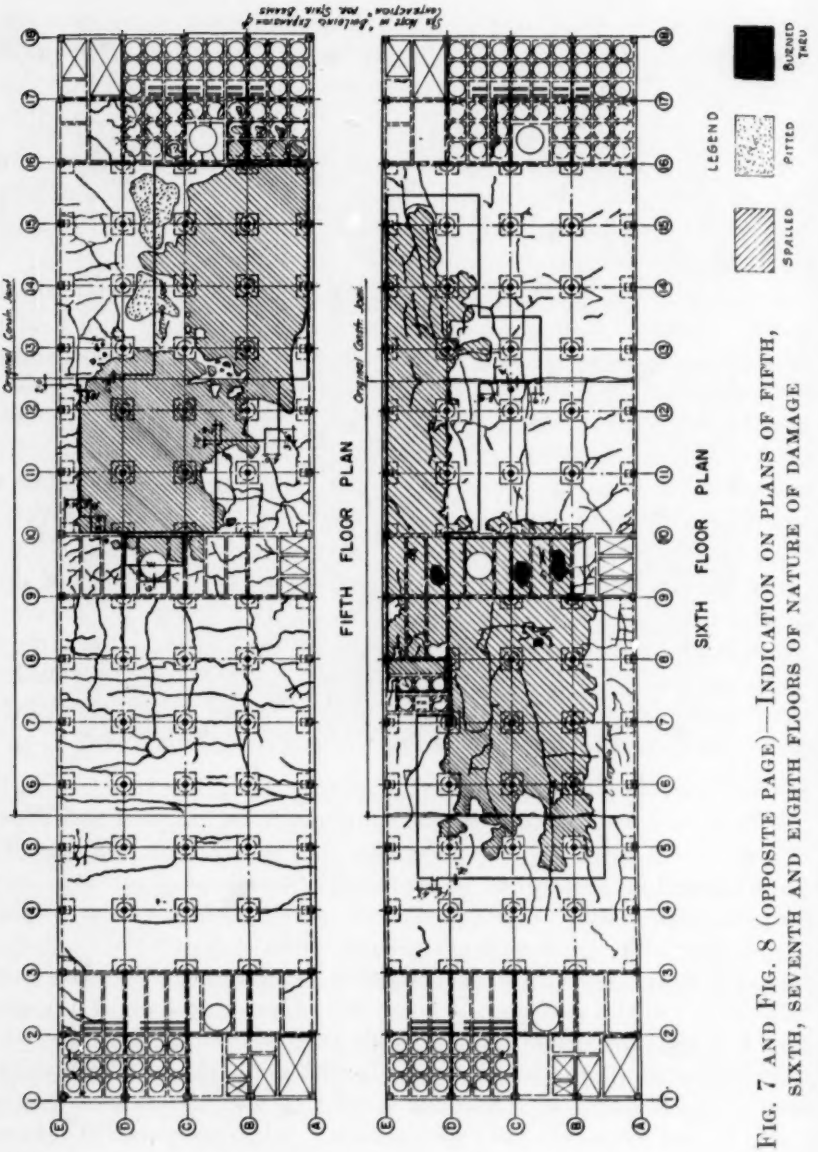


FIG. 7 AND FIG. 8 (OPPOSITE PAGE)—INDICATION ON PLANS OF FIFTH, SIXTH, SEVENTH AND EIGHTH FLOORS OF NATURE OF DAMAGE

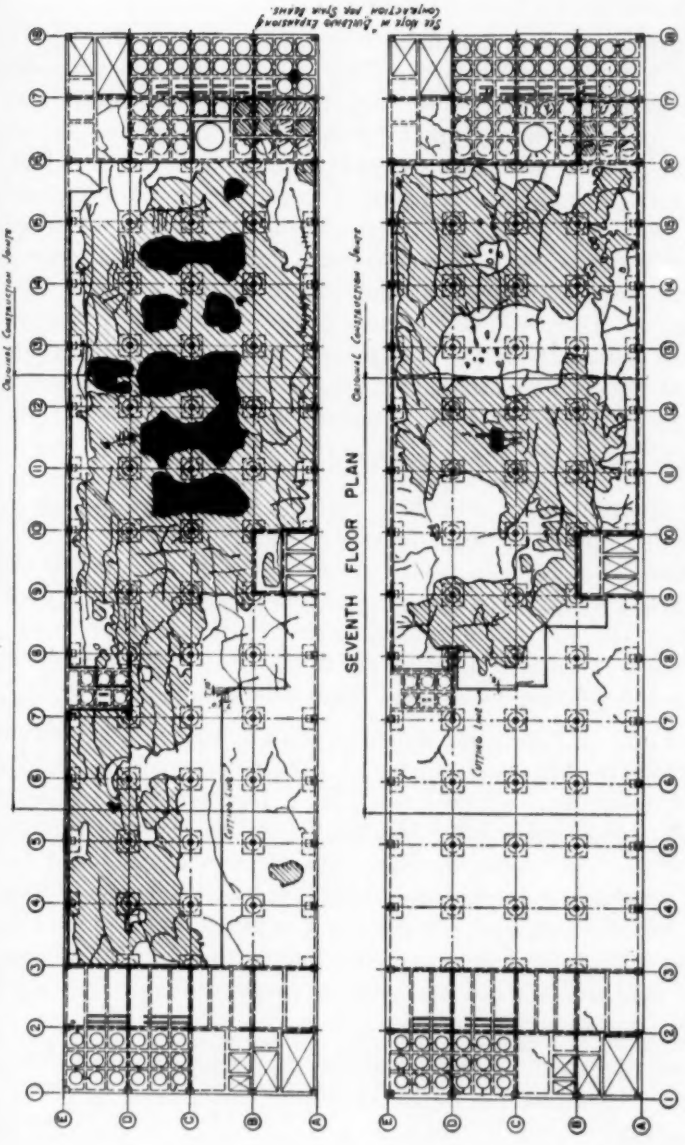


FIG. 8—SEE OPPOSITE PAGE

deep, to be explained by ordinary aggregate expansion, or unequal temperature in the slab. It is to be noted, that the older the building the less chance there is of such damage.

The aggregate used in the one-inch floor finish had about 45 per cent quartz, and 20 per cent limestone. We failed to find any place where the finish spalled. There were, however, a few places where 24-in. diameter "mounds" were formed by the finish lifting from the structural slab. These probably resulted from a more intense heating of local areas, or may have resulted from a combination of stress resulting from heating and the formation of steam under the topping. This absence of spalling of the floor finish might be due to some combination of these factors: (1) it took some time before it was exposed to the fire; (2) it was wetted down on top by the fire department (except on the seventh floor); (3) the smaller size aggregate ($\frac{3}{8}$ in. maximum); (4) the smaller amount of excess water ($3\frac{1}{2}$ gals. per sack, against $6\frac{1}{4}$ gals. in the structural member).

Cement and Concrete Strengths After Fire

In the seventh and eighth floor slabs, beams, and supporting columns, evidence of calcining appears. This is to be expected where temperatures of 1600°F. to 1700°F. or over, occur. The depth varied roughly with the length of exposure, and increased about one inch for every three hours of fire duration. Spalling of the lower section of the members, bared the upper layers to the heat, and in most of the areas it is difficult to get the exact time of exposure.

There seems to be a good deal of speculation as to what chemical action is involved in recalcination of hardened cement at high temperatures. However, we do know that a thin powdery surface was discernible immediately after the fire where the temperatures were over 1500°F. The concrete under that surface seemed hard to the geologist's hammer. Some three months later, it developed that the concrete was crumbly and powdery to a depth of as much as four to six inches. While this may have been due to air-slaking of the aggregate, the depth of the affected portion would probably indicate disintegration of cement compounds.

To learn the effect of additional wet curing on damaged concrete, cores were tested with the results shown in Table 1.

The immediate effects of wetting 24 hours appear negligible. Continued wetting, however, for 14 days, showed an increase in strength for the poorer specimens. The cylinders from areas less affected by the fire, showed the characteristic difference between the strength

TABLE 1
Strength in Pounds per Square Inch

Core Dry	Wetting 24 Hours	Wetting 14 Days	Wetting 14 Days & Air-Drying 14 Days	Percentage Change at Test
1802	1741	....	....	− 3
1802	....	2040	....	+13
1996	2219	....	....	+11
1996	....	2491	....	+25
2438	2382	....	....	− 2
2379	....	2532	....	+ 6
3547	....	....	3940	+11
2507	2518	....	....	+½
2909	....	2656	....	− 9
2692	....	....	3227	+20

of wet and dry cylinders.* The wetting and subsequent drying, however, showed considerably more increase in strength than would be expected. Some of the results omitted from the tabulation showed such high gains that we assumed an unfortunate selection of samples, which was eventually overcome by drilling two cores from each specimen. It is also interesting to note that a core taken from an area kept wet by the sprinkler during the fire showed a strength of 3547 p.s.i., while another core taken not over 18 inches away tested only 2438 p.s.i.

*Concrete tested in a wet condition is often found to have but 75 to 80 per cent of the strength of identical specimens tested when dry.

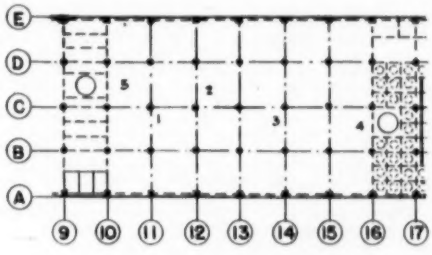


FIG. 9

FIFTH FLOOR TEST SPECIMENS (FIG. 9)

Point in Fig. 9	Concrete			Steel								
				Elastic Limit			Ultimate			% Elongation		
	Orig. Str.	Pres. Str.	% Loss	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.
1	4209	3043	28	50,430	48,880	−3	78,960	73,670	−7	23.7	25.0	+5
2	4209	3097	26	50,430	50,400	....	78,960	77,600	−2	23.7	26.0	+10
3	4209	3641	13	50,430	41,750	−17	78,960	76,220	−3	23.7	24.7	+4
4	4209	3695	12	50,430	49,690	−1	78,960	75,870	−4	23.7	25.5	+8
5				50,430	48,900	−3	78,960	80,490	+2	23.7	21.2	−11

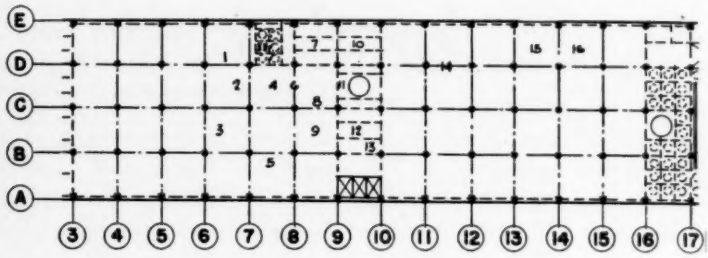


FIG. 10

SIXTH FLOOR TEST SPECIMENS (FIG. 10)

Point in Fig. 10	Concrete			Steel								
				Elastic Limit			Ultimate			% Elongation		
	Orig. Str.	Pres. Str.	% Loss	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.
1	4687	3297	30	54,350	50,430	−7	81,520	78,450	−4	20.7	....	no test
2	4687	2438	48									
3				54,350	46,610	−14	81,520	74,780	−8	20.7	20.7	....
4	4687	2712	42	54,350	43,300	−20	81,520	70,300	−14	20.7	22.0	+6
5	4687	3460	26	54,350	50,840	−6	81,520	80,490	−1	20.7	21.2	+2
6				48,400	46,800	−3	78,400	72,800	−7	21.5	25.5	+19
7				49,460	52,540	+6	77,170	80,080	+5	22.7	20.5	−10
8	4687	2627	44	48,400	46,800	−3	78,400	74,000	−6	21.5	24.5	+14
9				54,350	45,080	−17	81,520	68,620	−16	20.7	18.2	−12
10				49,460	51,640	+4	77,170	71,560	−7	22.7	20.0	−12
11				49,460	48,010	−3	77,170	77,000	....	22.7	18.7	−18
12				49,460	46,200	−7	77,170	72,020	−7	22.7	22.5	−1
13				49,460	53,990	+9	77,170	80,440	+4	22.7	22.2	−2
14	3926	2507	36									
15				54,350	48,900	−10	81,520	80,490	−1	20.7	21.2	+2
16	3891	2821	27	54,350	49,860	−8	81,520	76,940	−6	20.7	22.5	+9

Loss of Strength of the Concrete

Various specimens were cored throughout the area affected by the fire and were tested with the results tabulated. Exact temperatures and duration of exposure were impossible to obtain. It is also a difficult to say how much, if any, strength recovery took place, or to assign definite strengths to the original concrete. (See Fig. 13, A to D)

Effect of Fire on Reinforcing Steel

To determine the effect of fire on reinforcing steel, intermediate grade, as used in the building, tests were made with the cooperation of H. J. Cutler of the Bethlehem Steel Co. Bars were heated for four

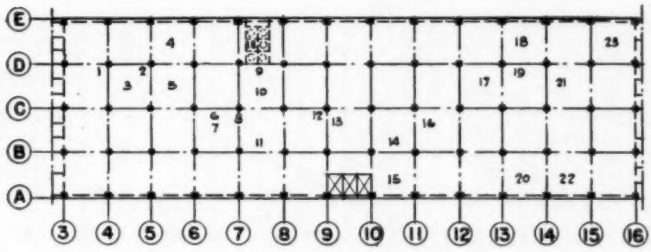


FIG. 11

SEVENTH FLOOR TEST SPECIMENS (FIG. 11)

Point in Fig. 11	Concrete			Steel								
				Elastic Limit			Ultimate			% Elongation		
	Orig. Str.	Pres. Str.	% Loss	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.
1	3282	2808	15	49,920	51,960	+4	78,450	79,470	+1	23.5	19.5	−17
2	3282	2334	29	52,170	57,710	+11	77,600	83,470	+8	23.2	20.7	−11
3				49,920	51,450	+3	78,450	81,510	+4	23.5	20.2	−14
4				49,920	40,760	−18	78,450	79,110	+1	23.5	22.2	−6
5	3282	2159	34	49,920	50,940	+2	78,450	79,320	+1	23.5	22.5	−4
6	3130	1996	36									
7	3130	2219	29									
8	3130	2807	10	49,920	52,470	+5	78,450	79,980	+2	23.5	22.0	−6
9	3130	2782	11	48,700	51,960	+7	75,600	78,960	+4	26.2	23.0	−12
10				49,920	49,670	−1	78,450	79,470	+1	23.5	24.2	+3
11	3130	3149										
12	3130	2698	14	52,170	47,280	−9	77,600	74,010	−5	23.2	20.0	−14
13	3130	2017	36									
14	3130	1389	56									
15	3130	2271	27									
16	3130	1867	40									
17				52,170	37,500	−28	77,600	57,500	−26	23.2	37.0	+59
18	3459	1860	46									
19	3459	1439	58									
20	3459	2507	28									
21	3317	1237	63									
22	3317	2753	17	49,920	51,540	+4	78,450	79,470	+1	23.5	24.5	+4
23	3317	2280	31									

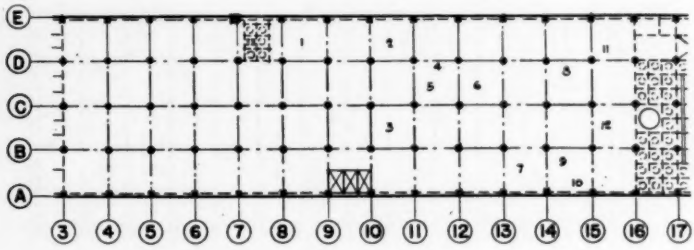


FIG. 12

EIGHTH FLOOR TEST SPECIMENS (FIG. 12)

Point in Fig. 12	Concrete			Steel								
				Elastic Limit			Ultimate			% Elongation		
	Orig. Str.	Pres. Str.	% Loss	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.	Orig. Str.	Pres. Str.	% Chg.
1				49,420	46,610	−6	75,920	78,450	+3	25.0	24.5	−2
2	4457	2245	50	49,420	52,980	+7	75,920	77,940	+3	25.0	22.5	−10
3	4457	2324	48	49,420	50,430	+2	75,920	75,390	−1	25.0	23.7	−5
4	4457	2181	51									
5	4457	1856	58	49,420	45,440	−8	75,920	71,830	−5	25.0	22.5	−10
6	4457	2430	45	49,420	50,940	+3	75,920	75,900	....	25.0	24.5	−2
7	4032	2694	33	49,420	48,390	−2	75,920	75,400	−1	25.0	23.0	−8
8	3820	2510	34	49,420	49,780	+1	75,920	76,050	....	25.0	22.2	−11
9	4032	2510	38	49,420	46,530	−6	75,920	77,280	+2	25.0	22.5	−10
10				49,420	49,160	−1	75,920	75,800	....	25.0	22.7	−9
11				49,420	48,850	−1	75,920	76,920	+1	25.0	21.2	−15
12	3820	2608	32	49,420	49,920	+1	75,920	76,920	+1	25.0	27.5	+10

hours to varying temperatures, and in another series, we varied the time of exposure from four to twelve hours, keeping the temperature constant. In all cases, the bars were air-cooled from the treatment temperature before being tested. For temperatures over 2000°F. the heating was done in sand to prevent scaling. Auxiliary tests were made to determine the effect on structural and high carbon bars to determine the value, if any, of changing the grade of the steel.

Intermediate grade—Fig. 13 shows a slight increase in elastic limit over the value as rolled, up to a temperature of about 800°F., then a gradual decrease as the temperature increases. The ultimate strength shows a slight gain up to a temperature of between 900°F. and 1000°F. and then drops off similarly, but at a slower rate than the elastic limit. The per cent of elongation as a measure of ductility, decreases till temperatures of between 700°F. and 800°F. are reached, then in-

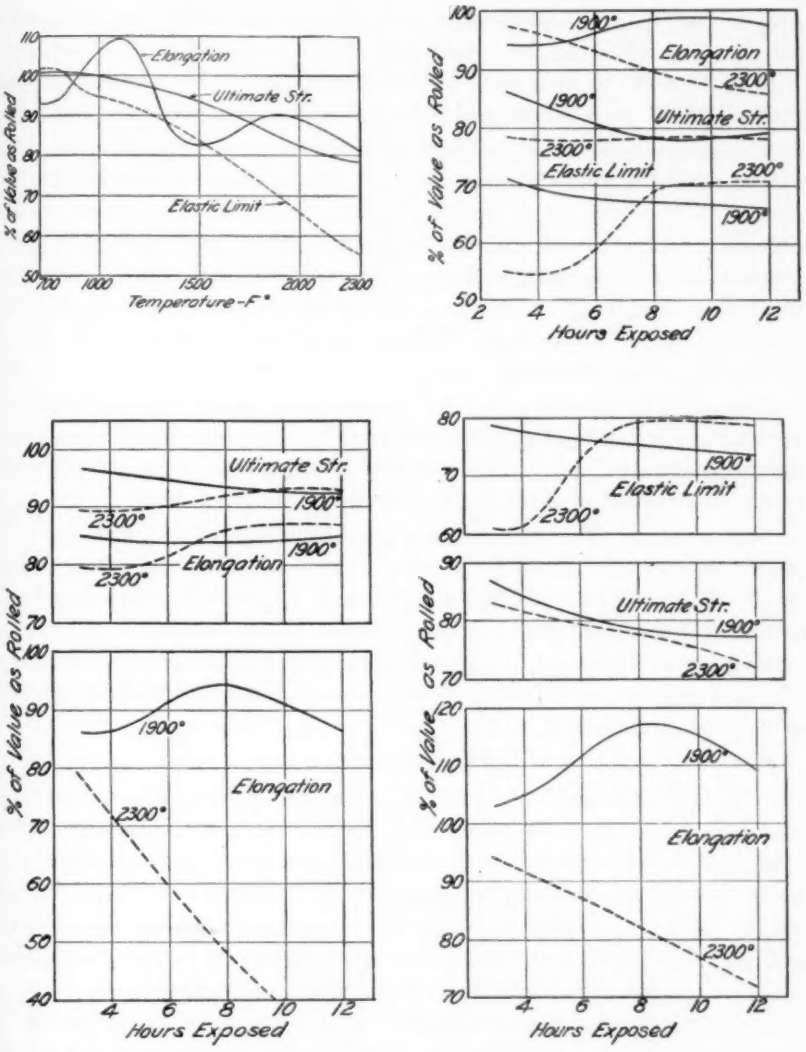


FIG. 13A (TOP LEFT)—EFFECT OF HEAT ON INTERMEDIATE GRADE STEEL, 4 HOUR EXPOSURE

FIG. 13B (TOP RIGHT)—EFFECT OF HEAT ON INTERMEDIATE GRADE STEEL EXPOSED 0-12 HOURS AT 1900° AND 2300°F

FIG. 13C (BOTTOM LEFT)—EFFECT OF HEAT ON STRUCTURAL GRADE STEEL EXPOSED 0-12 HOURS AT 1900° AND 2300°F

FIG. 13D (BOTTOM RIGHT)—EFFECT OF HEAT ON HIGH CARBON STEEL EXPOSED 0-12 HOURS AT 1900° AND 2300°F

creases rapidly to a temperature of slightly over 1100°F. as the rolling stresses are relieved, then decreases about as rapidly till a temperature of 1500°F. is attained. After this, a slight increase takes place up to 1900°F. and finally a decrease is noted as grain growth begins.

The figure also shows effects of varying time of exposure. The elastic limit for 1900°F. tests shows a slow decrease. At 2300°F., the center of the four hour sample showed little effect, as against the outside of the bar which showed considerable grain growth. At 8 hours exposure, the sample showed the coarse Widmanstaten structure uniformly throughout the bars. The ultimate strength showed little effect of increased exposure. Elongation, however, dropped off rapidly as the time was increased.

Structural Grade—The results as far as elastic limit and ultimate strength are concerned are all similar to but less pronounced than those for intermediate grade steel. For ductility the per cent drop was three to five times that of intermediate steel.

High Carbon—The elastic limit and the ultimate strength for high carbon bars were affected about the same as in intermediate grade steel. The ultimate at 2300°F., 12 hours exposure, shows a definite drop which was absent in the other charts. The elongation at 2300°F. occupies a position somewhat half-way between structural and intermediate grade. At 1900°F. there is an increase in ductility up to 8 or 9 hours of exposure.

BUILDING EXPANSION AND CONTRACTION

In fighting the fire, the outside of the building was kept wet and cold by the streams of water and the outdoor temperature. This made a difference in the expansion along the outside faces of the tile walls and the inside along the concrete skeleton.

The amount of expansion had to be measured and deduced from the movement of the tile walls and open cracks in beams and floors. In general, the upper sections of tile walls bonded with the beams below, thus leaving open vertical cracks. Some recovery probably took place before the readings were taken, as crushing of the tile in the plane of motion indicated and some allowance should also be made for the fact that the different floors did not have their maximum expansion at the same time.

A study of the after effects, coupled with observations made by Mr. Crosby, Nelson and others during the fire, and measurements taken immediately after the fire, the northeast corner of the building appeared to have moved east at the ninth floor 1½ in., eighth floor, 3 in.; seventh floor, 5 in.; sixth floor, 5 in.; fifth floor, 2½ in.; fourth floor, 1 in.; and

the southwest corner moved west at the ninth floor, ½ in.; eighth floor, ½ in.; seventh floor, 1 in.; sixth floor, 1½ in.; fifth floor, ½ in.; fourth floor, ¼ in.

The maximum expansion in the fifth floor slab took place slightly south of column row C; in the sixth, slightly north of the center line of column row D; and in the seventh and eighth floor slabs, at the center line of column row C.

At the first and ninth floors a crack opened across the entire width of the building between column rows 9 and 10 where, in the original construction, a section of the bay was removed for a service shaft.

The north wall lintel beams between column rows 17 and 18, and the south wall beam between column rows 1 and 2, (stair shaft beams) showed cracks at all floors between the third and the ninth. At the ninth and eighth floors of the north wall, the beam was sheared away from the supporting column. At the fourth floor level of the south wall, a section of the beam about 8 in. wide, and about 16 in. from the support was sheared through.

The north wall lintel at the seventh floor opened slightly over ½ in. between column rows 5 and 6 and about ¼ in. between column rows 12 and 13. At the sixth floor level, the north wall lintel between column rows 5 and 6 opened slightly over ¼ in. The recovery after the fire in these cases was almost 100 per cent.

The columns supporting the roof had horizontal checks on the inside face starting at a point three feet above the floor line and extending to six or seven feet above. The longitudinal roof lintels had two to three vertical cracks per beam.

While no definite information on the coefficient of expansion of concrete at high temperatures is available, no evidence of the separation of steel and concrete could be found for temperatures up to about 1100°F. Spalling made impossible any study at higher temperatures. A coefficient of .0000055 seems reasonably on the low side.

Assuming an average temperature increase of 900°F. on the sixth and seventh floors (which would be conservative) we should get an expansion of 900 x 316 x .0000055 = 1.56 ft. or about 19 in.

As far as any competent observations could be made, the expansion of the building at the sixth and seventh floors on the center line of column rows C and D, was at most 4 in. more than at the north wall; that is, the maximum building expansion was about 12 in. The difference between the theoretical and actual expansion of about 7 in. was obviously taken up by lateral distortion of the floor system, and compression in the floor concrete.

If it had not been for the spalling relieving the compression in the slabs, the damage to the columns and consequently to the building as a whole, would probably have been more serious than was actually the case.

The maximum observed expansion north and south was about three inches at column rows 11 and 12 at the seventh floor slab.

Vertically, the amount of expansion could not be determined. It was observed, however, that the steel bins not tied to the floor, raised the finish about 1½ in. higher than the nearby area on several of the floors. At the underside of the seventh and eighth floors, many column capitals, which were poured separately from the slab, were sheared off at the faces of the column cores.

It would be impractical to provide expansion joints to take care of the movement of a structural frame under fire conditions, and it is felt that their help in diminishing fire damage would be negligible. Furthermore, the uneven expansion of the various floors might even do more harm than good.

Tile Spandrel Walls

As the building frame expanded, it moved out not only the tile in contact with the skeleton, but perhaps a strip two to four feet away from the edge of the structural member. The outside face of the tile was kept cold all this time by the streams of water, and the outside temperature. This caused a good deal of cracking and some spalling of tile, breaking of mortar joints, and cracking along the joints of the web members and surfaces.

At the north wall between column rows 17 and 18, beginning at the third floor and going up through the ninth, the tile at the under side of the floor beams was crushed in the process of contraction. Some of the tile was ¾ to 1 in. away from its normal position before it was replaced. At the seventh floor lintel, more than half the tile was cracked.

There was also a good deal of damage at the entire southwest corner of the building. Two vertical cracks, the lengthening of which was easily noticeable during the fire, ran almost continuously the full height of the south wall. Other vertical cracks in the northeast corner were discovered next morning. The walls built in between columns showed crushing of tile at the planes of contact.

In the interior partitions, due to the top layer of tile bonding with the floor beams above, and the bottom layer with the floor below, vertical cracks opened, accompanied by crushing of tile in the horizontal joints. At the eighth floor lintel, between columns 16-A and

17-A, about half the brick work was pushed out ½ in. away from its original plane.

In many instances, when holes were drilled in the tile, preparatory to being replaced, a stream of water gushed forth—two months after the fire. This was rather general down to the third floor, and in the west wall down to the first floor. This water, entering damaged mortar joints, was imprisoned during the fighting of the fire, and settled at the various floor beams. Some spalling of the tile was due to the freezing of this water.

In this connection it is important to realize that some rain water got into the tile during the repair operations, especially at the north lintel of the seventh floor. It was impossible to keep the ends of the various pieces of tile covered with water-proof paper and plank, as was done in the original construction. Other tile still retains water poured during the fire and a watch will be kept this winter for tile cracking through either one of these sources.

Foundations

It was feared that the water used in fighting the fire would undermine the foundation mat. Exploration revealed some soil washed away between column rows E-17 and E-18, but this has been replaced by ramming concrete in place. Some water was bubbling up between columns E-5 and E-6, and between E-17 and E-18, but dyes placed in the water at the edge of the mat on column row A failed to appear anywhere else during a four-day test period.

REPAIR POLICY AND PROCEDURE

At the completion of the original concrete work, a check-up of the building disclosed a few curing checks at the west end of the second floor slabs. Outside of that, no visible cracks were found.

Concrete begins to lose strength, although slowly, at a temperature of about 200°F., and to repair all the areas affected would, to put it mildly, be impractical. Compromises had to be made to keep the repair cost within reason, and to get the job done quickly. In view of the cooperation of the Building Department of the City of Buffalo, it was decided that nothing should be done or requested that would in any way deviate from its regulations.

In general, the following were the determining factors in deciding upon the areas to be removed: (1) temperatures and duration of fire, such as to reduce strength of concrete, and reduce ductility and elastic limit of steel below safe values; (2) depth of spalling; (3) shear cracks; (4) location with reference to satisfactory construction joint, and last, but not least, (5) relative cost of several methods of repair.

It was desirable to obtain cores in the slabs as a check upon the work being done. The weight and size of a coring machine, however, presented many problems, especially when the floors to be cored were definitely in a weakened condition; the cost, prohibitive. Samples of the concrete and reinforcing steel were taken as the work progressed, and prepared for laboratory testing. While this prevented early planning, it caused no delays, since the design and details were being prepared on the job, occasionally with the help of the telephone.

Three methods of repairs to concrete buildings were available: (1) complete replacement; (2) the addition of fireproofed structural steel, or concrete members to those left in place, to carry either the full, or partial load; (3) removal of areas of damaged members, and replacement with comparatively non-shrinking, high compressive strength concrete.

The first method presents problems in taking care of shrinkage at joints in floors and columns, where the portion to be replaced is completely surrounded by old sections. The second method leads to larger members, and difficult connections.

The third method cannot be used to build up a member, where the compression section, reduced in strength, is left in place, or where the reinforcing steel was subjected to a high temperature, or where a straight shear connection is involved. In general the prime item in preparing a shear connection, or for that matter any connection is (1) the roughening of the surface, (2) the mechanical adhesion of the binder (mesh or reinforcing steel) to the old concrete, and of the new concrete to the mesh, and (3) the strength of the concrete to which the material is added. This method has the advantage of ease in handling, where small isolated areas are involved, or where a repair of this type will save costly removal of members and sometimes consequent bracing and shoring.

A difficulty involved with gunite is the problem of matching finishes of poured concrete. A finish coat of about $\frac{1}{4}$ in. has to be applied in comparatively wet consistency, producing shrinkage cracks.

A combination of all three methods was used.

Column Heads

The most serious problem on the job involved the repair of column heads. The number of them spalled was considerably in excess of expectations. Cracks were explored and cuts made to sound concrete. From sketches and plan of parts remaining, the repair design was developed and checked for bearing on the bracket and diagonal tension in slabs. Some allowance was to be made on the resistance to shear

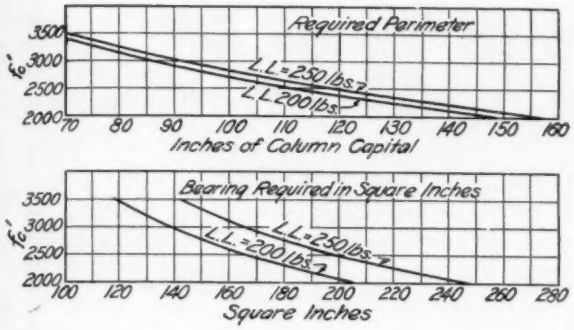
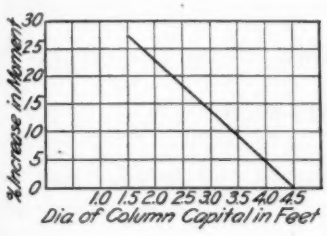


FIG. 14



EXCESS PER CENT OF MOMENT RESISTANCE IN PRESENT DESIGN

Bay	Band	200 Lb. L.L.	250 Lb. L.L.
Interior	Col. Pos.	10.4	2.73
	Col. Neg.	9.4	5.00
	Mid. Pos.	9.25	6.30
	Mid. Neg.	9.25	6.30
Disc. End	Col. Pos.	6.06	2.70
	Col. Neg.	14.75	5.54
	Mid. Pos.	8.80	6.52
	Mid. Neg.	22.20	19.60

of the bond between the old and the new concrete. The steel area was then adjusted on the basis of the reduced column head size. Charts were prepared (see Fig. 14) as follows:

1—The required perimeter of column capital for various values of f'_c. To obtain points on the curve, it was assumed that the remaining heads were all circular. The resistance diagonal tension was taken at a section d-1½ in. from the assumed circumference.

2—The bearing area required on the column head was figured for the total load on the bay for various values of f'_c. It is to be noted that the column heads were poured separately, and a plane of cleavage exists at that point.

3—This curve gives the increase in M_o for various values of c.

When it came to the actual working out of the project, it was found that wherever the column heads spalled the concrete inside was in bad shape. This was to be expected as the hottest portion of the fire was near the top so that, as a general rule, where the column head spalled badly, the column capital had to be removed completely.

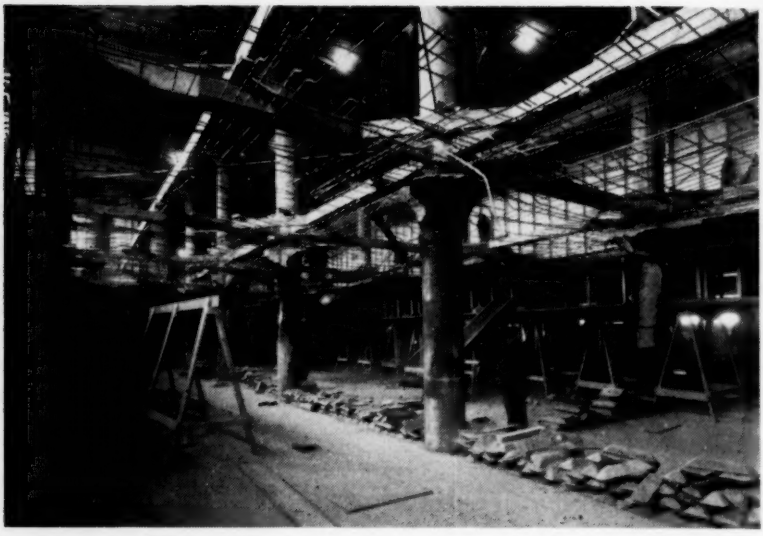


FIG. 15—WEST END SEVENTH FLOOR, CUTTING COMPLETE PRELIMINARY REPAIR WORK

Load Tests

To establish a standard of comparison a bay in the third floor (designed for a live load of 250 lb. per sq. ft.) and a bay in the fourth floor (designed for a live load of 200 lb. per sq. ft.) were tested with the results shown in Fig. 16. Only the deflection at the center of the bay is given.

The third floor bay had a construction joint through the center. Since no cracks developed, and no more pig iron was available for two days, this test was carried to but 370 lb. per sq. ft. superimposed load; the fourth floor to 300 lb.

To determine the effect of the cracks caused by the fire, a bay on the sixth floor, which was on the border of the area to be cut out, designed for the same load as the one on the third floor was also tested. It was found that for the same loads the deflection was greater, the final recovery less in the fire affected floor and new cracks began to appear at a live load of 275 lb. per sq. ft.

It has been found that:

1—Ordinary construction joints across center lines of bay are not harmful for uniform loading. This is important, as we have additional joints where new work joins old.

2—Ordinary irregular cracks, while unsightly and poor for future insertion of bolts increase the slab deflection, reduce recovery but do

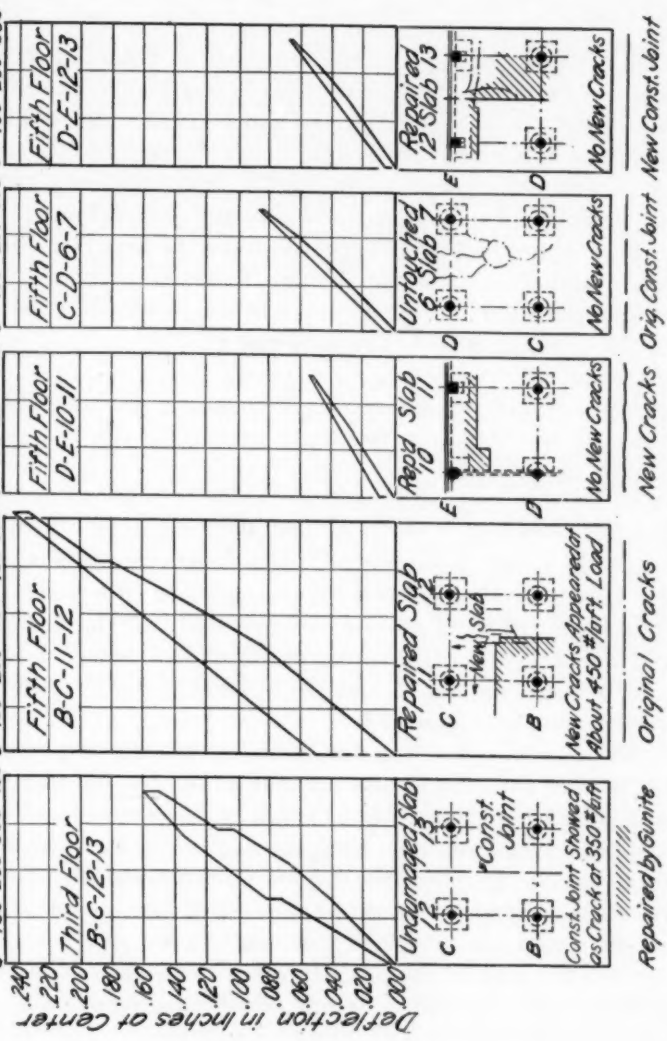


FIG. 16—FLOOR LOADING TESTS

not necessarily reduce the carrying capacity below what is considered safe.

Materials for Rebuilding

In general materials used for rebuilding were as far as possible the same as those used in the original structure.

The most serious problem presented itself in obtaining minimum shrinkage of newly placed concrete when it was poured against joints on all four sides. It was decided to use a low alumina high iron cement which is marketed in New York State under the designation 15-C. It appears to have the lowest shrinkage of any commercial portland cement. The original aggregates while not satisfactory from the point of view of fire resistance appeared to be the best from the point of view of shrinkage. Efforts were made to keep out chert.

Limestone gravel, $\frac{3}{8}$ in. maximum, was used to replace the spalled shells of the columns, where they were poured.

Plan of Operations

An examination of the damage to the building disclosed that: (1) the damage to the fifth floor was least serious; (2) the area damaged on the sixth floor was in practically every case over portions of the fifth floor that were in good shape; (3) the seventh floor was the most damaged, necessitating practically complete removal. It was also feared that the columns supporting the seventh floor might be more seriously damaged than a general examination revealed; (4) the eighth floor was damaged only in the east half; (5) the beams along the north wall of the seventh floor was practically all damaged seriously; (6) beams between the fourth and ninth floors at both southwest and northeast corners were damaged.

To make the most speed with minimum expense, cutting was started on the fifth floor with the idea of starting at the bottom and training an organization where the problems might be less serious, and working on a floor that would not be entirely opened up. This last point was essential in view of the fact that the building was still shrinking.

It was then decided to jump to the seventh floor so as to get the most serious problems out of the way and to allow two working platforms for the finishing of the job. In other words, after the fifth and seventh floors were completed, it was possible to remove and replace the sixth and eighth floors without any interference. The portions of the sixth floor that were weak could be shored down to the fifth floor, to support the seventh floor work.

Equipment

A twin platform tower was set up on the south side of the building, between bays 10 and 11, as that location was just off the driveway

from Michigan avenue, and near the center of operations. Along one of the towers a chute was arranged for dumping the debris on trucks. A half yard bucket was attached to the cable in the other, for raising the concrete to the proper floor, where it was dumped into a movable receiving hopper. Steel and timber were raised through this tower while no concreting was in progress. The concrete was delivered to the job in transit mixers. Since most of the operations were confined between the sixth and eighth floors, a hoist having a speed of 250 ft. per minute was rented. One of the freight elevators was in practically constant use bringing up material that came in cars into the building, and transferring items between floors that were not too long for the cage. A hand shear and bender were set up to do emergency reinforcing steel fabrication on the job. It was planned to provide air for a maximum of

5 concrete breakers	@ 46	=	230
4 clay diggers	@ 33	=	132
6 chippers	@ 28	=	168
			530 cu. ft. per minute

Since we decided to operate on less than the maximum pressure, and the kind and number of tools varied from day to day, a 500 ft. compressor was used with very satisfactory results. A separate compressor was furnished by the gunite subcontractor for the limited time he was on the job.

It was difficult to keep the feed on the air line lubricators adjusted due to the varying number of tools, and it was therefore decided to oil each tool separately as required, even though it resulted in some loss of time.

A two inch pipe was run up along side of the building from the compressor, and ¾-in. hose was connected to the pipe at each floor where the work was being done.

Movable scaffolds were hung from a permanent trolley beam attached to the roof beams. These were used for cleaning the walls and for outside cutting and repairing.

Method of Cutting

Before cutting operations were started, two lines of shores were erected, one 3 in. back of the final cutting edge and the other about 2 ft. outside of that line. This was done in order not to open up the cracks in such concrete as was to remain, due to the cantilever action of the hammers on the slab being cut out. Additional shores were placed under floor cracks in the slab remaining, in order to avoid their opening up and spreading during the cutting.

A platform six feet high, consisting of 2 in. planks placed on horses about 6 ft. on center was erected, upon which the cut material was dropped. This simplified the cutting operation and the removal of the material. At times when the cutting was progressing more rapidly than figured, or cutting had to be done at more widely distributed points, a 2 in. plank was placed on the floor below, and the material dropped upon that. This, of course, could not be done where the cutting was in large pieces, or where the floors below were in bad shape.

Operating inside the building we found that the maximum pressure that could be supplied at the floor without causing too much vibration for the remaining slab, was anywhere from 50 to 60 lb. This reduced the efficiency of the tools, designed to operate at about 80 lb.

We operated 55-lb. concrete breakers for cutting out the middle bands. Twenty-three lb. clay diggers with chisel points were used for cutting the balance of slabs and drop panels to within 3 in. of the final cutting line. Eleven lb. chippers with chisel points were used to get to the finish lines.

In those slab areas where new construction joints were to be made in the center of the bay, an 11-pound retainer type chipper with chisel points was used for underscoring the underside of the floor slab to a depth of about one inch. To drill holes for the dowels in the slab, we used a jack hammer equipped with $\frac{7}{8}$ in. bar and $\frac{7}{8}$ in. bit—the smallest size obtainable with a central hole for blowing out the material.

Normally the points were made from 9-in. blanks. For cutting over the column capital, however, we used 12-in. blanks in order to be able to get between and below the reinforcing steel. The material was cut from above to about two or three inches below the reinforcing steel. The final cutting of the column capitals was done from a platform below.

Effect of cutting vibration on freshly placed concrete

To determine when to place concrete, it was decided to test the effect of the vibration, caused by adjacent cutting, upon crushing strength and bond resistance of the new concrete. Five 6 x 12 cylinders, and five specimens 4 x 4 x 12 in. long with a $\frac{1}{2}$ in. plain round inserted, were cast, and tested with these results.

Specimen	Change in Crushing Strength	Change in Bond Resistance
1	Base	Base
2	+ 7 per cent	—30 per cent
3	+18 per cent	—14 per cent
4	Base	Base
5	0 per cent	+46 per cent

Specimens No. 1 and 4 were controls left undisturbed in the construction office; No. 2 specimens were poured at 4:30 p. m. and left undisturbed till 8 a. m. next morning, when they were placed 1½ bays away from where the cutting operation took place; No. 3 were kept undisturbed for 2½ days, and placed on the forms 2 bays away from where the cutting took place; No. 5 were placed on the forms 2 bays from where the cutting took place immediately upon being poured, but were subjected to vibration from nearby sources for 8 hours only.

As the cutting proceeded the position of the specimens with relation to the cutting lines varied as would normally occur on the job. Every effort was made to duplicate job conditions in pouring and curing.

As far as crushing resistance is concerned, the specimens placed immediately on the forms while the cutting was going on showed practically no change, the ones subject to vibration after 15½ hours of rest showed an increase of 7 per cent, and the ones that had approximately 2½ days of curing showed an increase of 18 per cent in crushing strength. All the specimens were 16 days old when tested.

With reference to bond resistance, however, the specimens poured on the forms while cutting was going on showed an increase of 46 per cent, while the one that had 15½ hours of curing before being placed on the forms showed a loss of 30 per cent, and ones that had 2½ days of curing showed a loss of 14 per cent.

Since no accurate gauging of the curing or the time or intensity of vibration was used, no general conclusions should be drawn from these tests. For this particular job, it was decided to pour concrete only at the end of the week and this only when not more than one day's cutting on the floor remained to be done. Otherwise the cutting operations were kept at least 3 bays away from where new concrete was being placed.

Bracing

Immediately after the fire, when the debris was removed, 8 x 8 struts were placed in the area bounded by column rows 10 and 16, on the column center lines immediately above the seventh floor slab. This was deemed sufficient as long as nothing was disturbed, and the building was shrinking. Immediately upon the letting of the contract, however, these struts were replaced by the system of bracing shown in Fig. 17. This gave us at least some tie across every floor in the building.

Since the building was still shrinking, and 1 in. rods were the maximum size immediately available, they were used. All tie rods were put in with an initial tension. Extensometer readings on the few

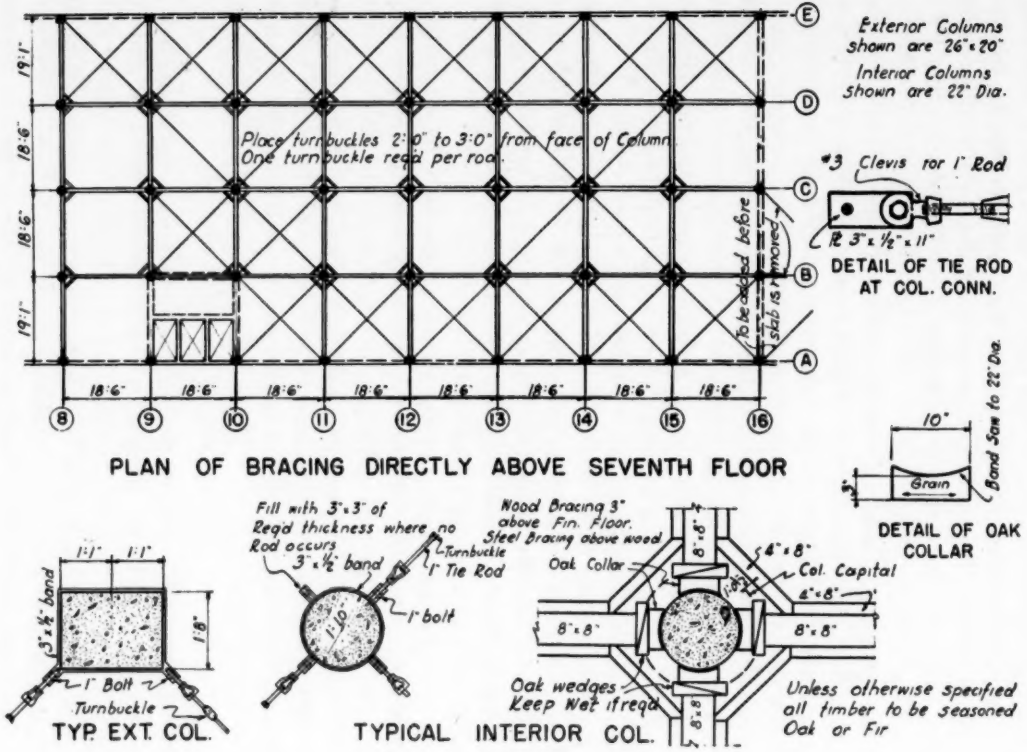


FIG. 17—PLAN AND DETAILS OF BRACING

rods, where the marks were not obliterated, showed the bars to be slightly in compression within two weeks of the time they were placed. The oak wedges did not have to be tightened until almost a month after repairs started.

By wedging, the ½ x 3 in. steel bands were reusable on the upper floors. As the tie rods were made in one long and one short piece, the short ends only were scrapped.

The location of the wood bracing was such as to make possible economical cutting of column capitals, and to permit the placing of shoring, if and when it should be required.

Before cutting was started on the fifth floor, bracing was erected. By the time the braced portion of the seventh floor was cut out, the fifth floor concrete was a week old, and the material was moved to the west end of the seventh floor. By the time the eighth floor was cut out, the seventh floor below had been replaced although the bracing in the east end was still left in place. And since the sixth floor in the area below was in fair shape, and the ninth floor above was not damaged, it was decided to omit the tie rods on this floor, except where interior columns or column capitals were being removed. It was done primarily to save the cost of breaking and repairing an ornamental brick belt course at that elevation. Before cutting started at the east end of the sixth floor, there was sufficient bracing material available from the east end of the seventh floor.

Since the bracing was now removed from the seventh floor, and no new material had to be purchased, and since the shrinkage of the building had definitely ended, it was decided to use tie rods as well as struts on this floor. Before work was started on the west end, the bracing system from the area above, and eighth floor was available.

REPAIR OPERATIONS IN DETAIL

Fifth Floor

The figured temperatures and duration of the fire showed that the reinforcing steel could be reused. Those areas where little or no spalling took place had protected the upper section of the concrete members from loss of strength, and could be reclaimed by chipping about 1 in., in some cases about an inch above the steel bars, and then guniting new concrete. The concrete and reinforcing bar tests substantiated the decision. Some of the drop panels cracked on section starting at the corners. The loose pieces were removed, and replaced with gunite. The northeast section of the floor, even though weakened slightly, and cracked, was left to help provide a base for the operations on the floors above, thus speeding the repairs.

The area cut out is shown on the crack survey (Fig. 7). It will be noticed that in a few cases the construction joint was not placed on the center line of the bay. This was due to the fact that a construction joint had already existed in the form of a crack, and it was decided that a new construction joint at the point would not be any weaker than the present crack and accordingly the concrete was cut back to that line. To help develop the shear at those points, dowels were provided.

In cutting along the beam the joint was made so that when shrinkage takes place the crack would be under the wall and would not show in the floor space.

Practically all the reinforcing steel had to be saved, since it was anchored to concrete remaining in place. Bars were supported from joists resting on top of the bracing. The forms were then erected about two feet below their final elevation and five adjustable shores were used per bay to hold them temporarily in position as the bars were raised and brought to their nearly correct position. As the cutting was completed the forms were gradually raised to their final position and the bars straightened out as well as possible.

Rods that were in bad shape and not anchored into the concrete at either end were removed where that was feasible and left in place where the problem of removal was difficult. In either case new bars to carry the load were provided. Anchored bars which had to remain were spliced with new steel for the length of the distorted portion. The new steel was welded to the old bars for a distance of about 3 or 4 inches at both ends, or spliced where possible.

The repairs to the damaged column capitals supporting the fifth floor are shown in Fig. 18. In general, pockets for seating the new concrete were made and slots were cut to permit the pouring of concrete at the bottom of the capital where the top remained intact. They were then poured with the floor. This method of procedure was used: (1) roughen surface thoroughly; (2) wet surface; (3) apply coat of neat cement paste; (4) place concrete with not more than 3½ gal. water content; (5) vibrate concrete; (6) place surrounding floor area; (7) vibrate again after a half hour (it takes about a half hour before the concreting operation reaches the column capital).

This procedure gave excellent results where patches of concrete had to be placed. The gunite subcontractor was given permission to try to break the bond at any one of these patches, and he failed to do so. The break in every instance was back in the old concrete.

Where a large section was added, or where portions of the capital had to be added around the entire periphery, this procedure was not

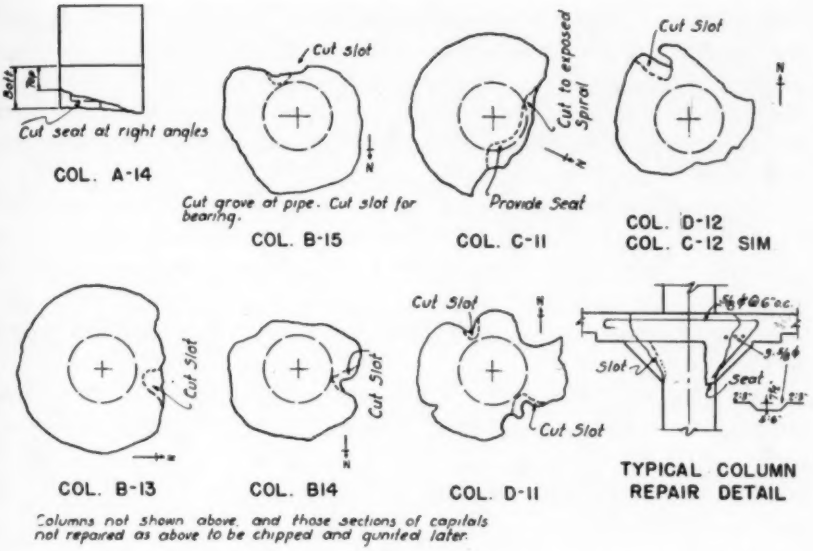


FIG. 18—DETAILS FOR REPLACING DAMAGED PORTIONS OF COLUMN CAPITALS

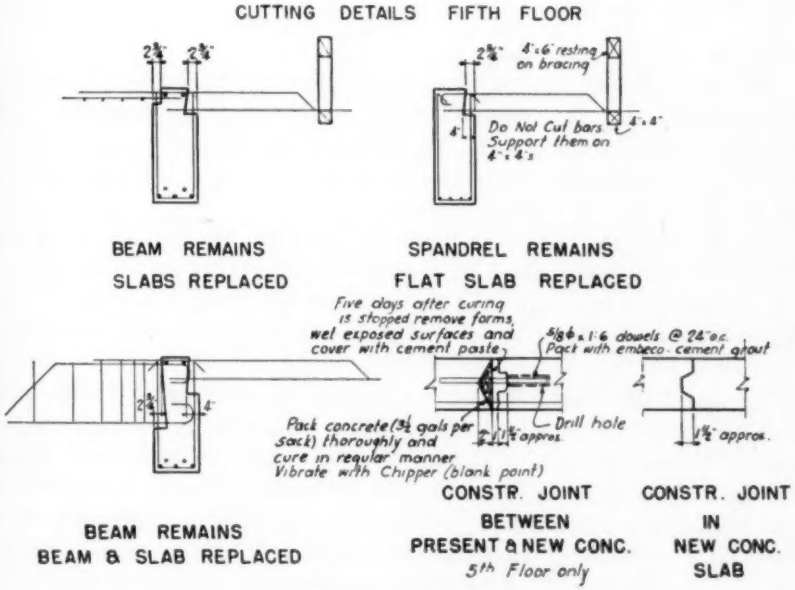

CUTTING DETAILS FIFTH FLOOR
4"x6" resting on bracing
Do Not Cut bars Support them on 4"x4"s
BEAM REMAINS SLABS REPLACED
SPANDREL REMAINS FLAT SLAB REPLACED
Five days after curing is stopped remove forms, wet exposed surfaces and cover with cement paste
Pack with embeco-cement ghout
Drill hole
Pack concrete (5½ gals per sack) thoroughly and cure in regular manner Vibrate with Chipper (blank point)
BEAM REMAINS BEAM & SLAB REPLACED
CONSTR. JOINT BETWEEN PRESENT & NEW CONC. 5th Floor only
CONSTR. JOINT IN NEW CONC. SLAB

FIG. 19—BEAM AND SLAB REPAIRS

necessary, as the ordinary concrete (about 5 in. slump) we used made a perfect bond, after the surface was roughened. We did, however, go back and vibrate the concrete for a second time, at all old joints. That is, we placed the sections against the old concrete first, and when the regular pouring operation reached the old sections, the concrete was again vibrated.

Measurements of the shrinkage of concrete were taken after the curing was stopped, and the amount was so small that it could not be discerned in a length of 100 feet. It is obvious that the amount of shrinkage will increase, but since the packing could not be longer delayed, it was decided in the future to eliminate the special key shown in Fig. 19.

A test cylinder was made with concrete used in packing the joints prepared in the same manner as that going into the joint. This concrete tested 2900 p.s.i. at the end of three days. It was decided to remove all the shoring and bracing at that time, as the slab itself was almost three weeks old.

The gunite operation started after all the forms were removed. The section of the slab remaining in place east of column row 10, and west of column row 16, along with the beams in the adjacent bays, as well as the columns in the area affected by the fire were repaired by gunite.

Cutting details for the fifth floor, which were more or less followed for all floors, are shown in Fig. 19.

Gunite

We used 1:3 mix instead of the customary 1:4, to reduce the amount of rebound and to cut cost in cleaning machinery and equipment. The mesh was standard 2 x 2, 12/12, galvanized, fastened to the concrete with ¼ in. expansion bolts placed 18 in. on center for ceilings, and 24 in. on center for vertical surfaces.

To improve the finish as much as possible, the gunite work was stopped about ½ in. short of the final surface and a fine brick sand was substituted to enable the troweling of the finished surfaces. The use of lime, or the addition of more water would perhaps have improved the finish but one would reduce the strength, and the other would cause shrinkage cracks. We were perhaps unfortunate, in that local union regulations do not permit regular plasterers to do this kind of work. When the gunite machinery was busy, or where the repair was at an especially isolated location or where damage to machinery was feared, the surface was prepared in exactly the same manner as for gunite, but the repair procedure followed was as previously described in steps 2 to 7. Vibration was done with the blank point on a concrete clay digger or chipping hammer.

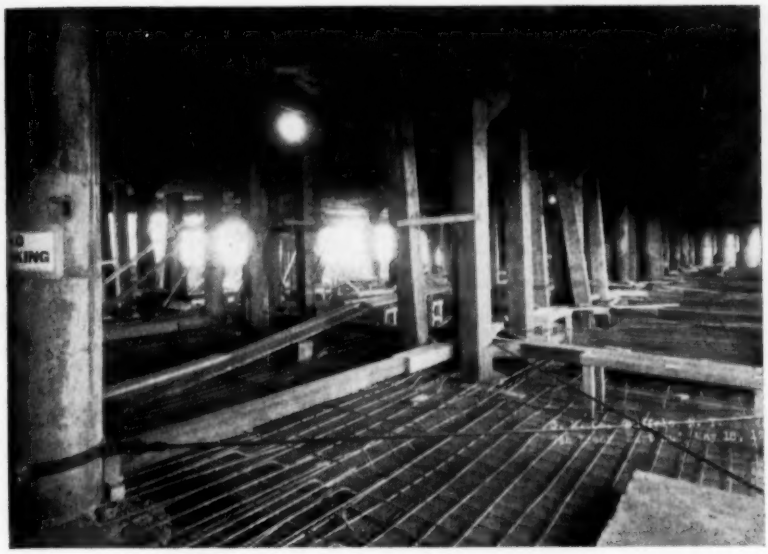


FIG. 19—SHORING UNDER WAY EAST END; SEVENTH FLOOR READY FOR CONCRETE

Seventh Floor

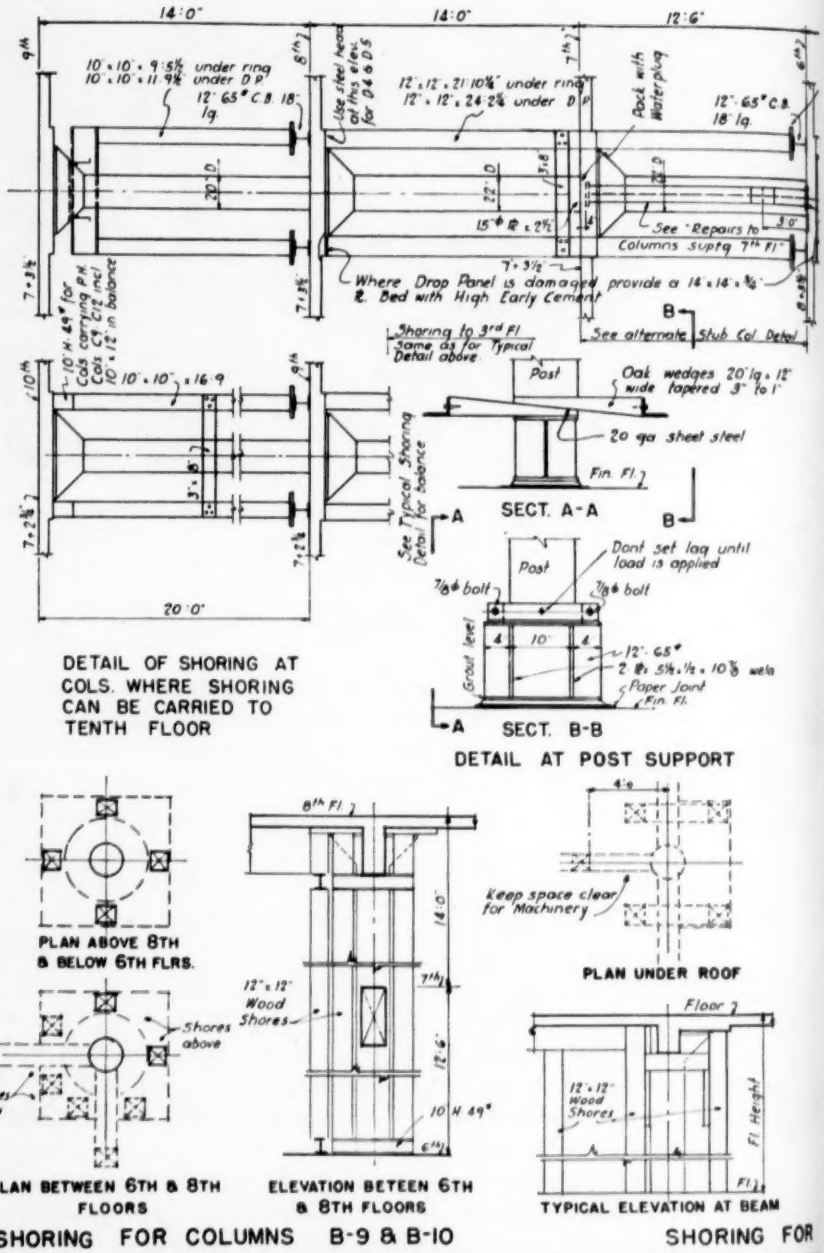
The cutting lines for the seventh floor are shown in Fig. 8, and special details are shown in Fig. 25.

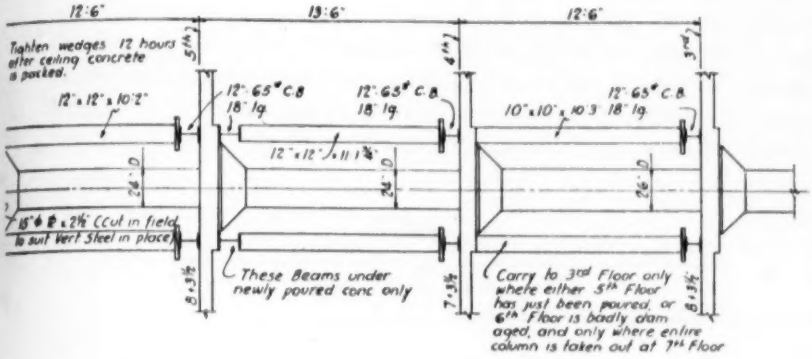
As the cutting operations started, it soon became evident that the effect of the fire on the columns supporting the seventh floor was much more serious than had been anticipated. This was especially true of the column capitals and the upper portion of the interior columns, and the inside faces and column capitals along the north wall.

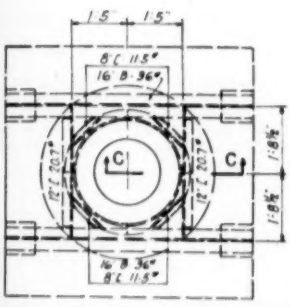
Three schemes for replacing the interior columns were considered. In all of them the floor had to be shored. In the first two methods the present column was to be removed completely:

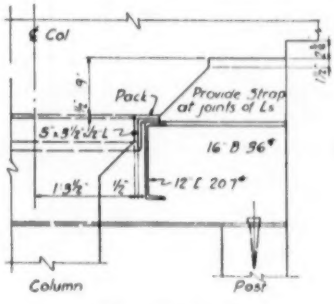
(1) Pouring complete new columns and capitals with the floors. This method, because of the necessary shoring, would leave four holes in the drop panels per column, or a total of at least 120 holes in the seventh floor slab to be patched later. It would also make it almost impossible to prevent settlement when the new columns took all the shrinkage; would slow down operations, or leave a substantial portion of the building on shores at one time.

(2) Placing steel columns inside of the core to take the shoring load. This method obviates the difficulties mentioned above, but brings in another one, of providing sufficient bearing area within the


14'-0"
14'-0"
12'-6"
10" x 10" - 9.5% under ring
10" x 10" - 11.9% under D.P.
12" 65# C.B. 18' lg.
Use steel head of this elev. for D4 & D5
12" x 12" - 21.10% under ring
12" x 12" - 24.2% under D.P.
Pack with Waterplug
12" 65# C.B. 18' lg.
20'-0"
22'-0"
15-¾ ø x 2½
See "Repairs to Columns supt'g 7th Fl."
Where Drop Panel is damaged provide a 14" x 14" x ⅜" ø Bed with High Early Cement
10-¾ H. 49# for Cols carrying P.H. Cols C1 C12 incl 10" x 12" in balance
10" x 10" x 16'-9"
Shoring to 3rd Fl Same as for Typical Detail above
See alternate Stub Col. Detail
Post
Oak wedges 20" lg. x 12" wide tapered 3" to 1"
20 ga sheet steel
Fin. Fl.
SECT. A-A
See Typical Shoring below for balance
Dont set lag until load is applied
⅞ø bolt
⅞ø bolt
Grout level
12" 65#
2 ø 5½ x ½ x 10⅜ weld
Paper Joint
Fin. Fl.
SECT. B-B
DETAIL AT POST SUPPORT
DETAIL OF SHORING AT COLS. WHERE SHORING CAN BE CARRIED TO TENTH FLOOR
PLAN ABOVE 8TH & BELOW 6TH FLRS.
Shores above
Shores below
PLAN BETWEEN 6TH & 8TH FLOORS
8th Fl.
14'-0"
12'-6"
12" x 12" Wood Shores
10-¾ H. 49#
ELEVATION BETEEN 6TH & 8TH FLOORS
Keep space clear for Machinery
PLAN UNDER ROOF
Floor
12" x 12" Wood Shores
Fl. Height
Fl.
TYPICAL ELEVATION AT BEAM
SHORING FOR COLUMNS B-9 & B-10
SHORING FOR

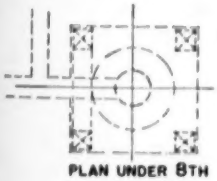

12'-6"
13'-6"
12'-6"
Tighten wedges 12 hours after ceiling concrete is packed.
12" x 12" x 10'-2"
12" 65# C.B 18" lg
12" 65# C.B 18" lg
12" 65# C.B 18" lg
10" x 10" x 10'-3"
12" x 12" x 11'-3"
24"-0"
24"-0"
26"-0"
15"ϕ ℄ x 2½" Ccut in field to suit Vert Steel in place
These Beams under newly poured conc only
Carry to 3rd Floor only where either 8th Floor has just been poured, or 6th Floor is badly damaged, and only where entire column is taken out at 7th Floor
TYPICAL SHORING DETAIL



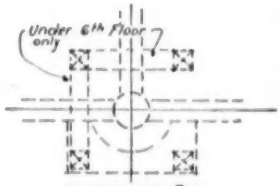


PLAN AT 9TH FLOOR FOR D9 - D13 INCL.
PLAN AT 8TH FLOOR FOR D4 & D5

SECTION C - C



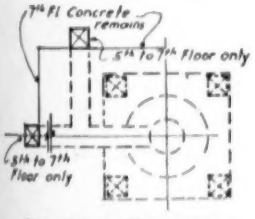
PLAN UNDER 8TH & 9TH FLOORS



PLAN UNDER 5TH & 6TH FLOORS

Columns supporting 6th & 8th Floors

Note: To replace columns supporting 8th Floor shore in same manner as shown, but stop at 5th Floor level. For Columns supporting 6th Floor, use stub columns and shore down to 4th Floor only.



PLAN UNDER 7TH FLOOR
COLUMNS D-7 & D-8

FIG. 21—SHORING DETAILS

vertical column bars to take the load. Assuming that the vertical bars are placed perfectly, they would permit of a maximum bearing plate diameter of 22 in.—7 in.—15 in. If the bars from the floor below, or the bars near the floor above are bent out of position or misplaced, the size of the plate would be considerably reduced. With this type of design, in accordance with building code requirements, and even ordinary good practice, the dowels of the sixth floor, and

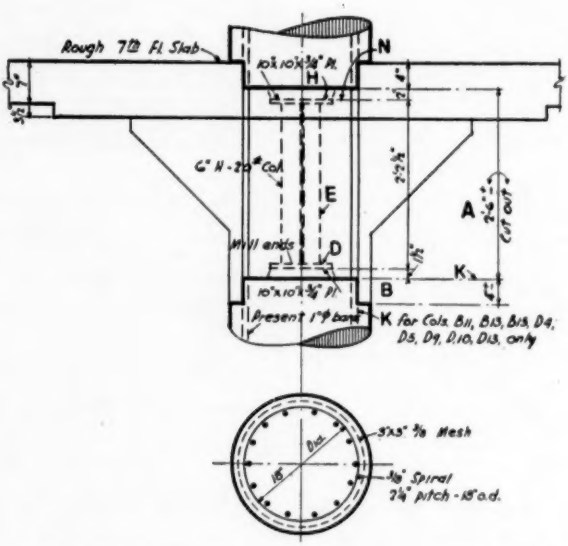


FIG. 22—REPLACEMENT OF COLUMN CAPITALS

Supporting 8th Floor Cols. B11, B12, B14, C10, C12, C14, C15, D9, and D10.
Supporting 7th Floor Cols. B11, B12, B13, B15, C6, D8 to D15 Incl., D4 to D6 Incl.
Supporting 6th Floor Cols. B8, C7 and C8.

Operations

A—Cut out 2′-6″ of column as indicated leaving vertical reinforcing steel intact.
B—Cut 4″ ± shoulder as shown.
C—Collapse exposed spiral to drop at *B*
D—Bed under and level base plate *D* with water plug and wait ½ hour.
E—Place 6″ *H* column after inserting it between bars.
F—Put plate between bars on top of 6″ *H*.
G—Replace spiral and wire to bars.
H—Pack above plate *H* with water plug.
K & M—For cols. B11, B13, B15, D4, D5, D9, D10, and D13 only.
K—Cut out soft concrete inside of spiral below line *K*.
L—Pour column head to underside of drop panel.
M—Pack or gunite column below capital.
N—Pack space *N* with grout and keep damp for 2 days before slab is poured.
O—Shores can be removed after operation *M*.

dowels for the seventh floor columns will have to be saved and vertical steel and spirals of the same size and area as furnished at present would have to be replaced. This solution has the one big advantage that it obviates the difficulty of determining the loss of strength in the columns subjected to the fire.

(3) Taking out the column capital only and inserting a steel column to take the shoring load. In using this method, we could reduce the shoring load by taking more than half of it on the vertical reinforcing steel. This could only be used, however, where the column section below is strong enough to carry the load, or where repairs to make it so, would not be too costly.

Method 2 was decided on for column row C, that being the most seriously damaged throughout the height of the member. When operations were started it was discovered that the bearing plates would have to be reduced in size considerably, and that it was a very difficult task to get the concrete around the big H section. To prevent overstressing the bearing, the shores were kept in place for two to three days until new concrete was placed to the under side of the drop panel. This meant that some of the load was transferred onto the reinforcing steel, and accordingly the bearing stresses were relieved.

It was decided, therefore, that wherever the column had to come out in its entirety, to modify this procedure and use a combination of methods 2 and 3, using 6-in. H columns to take only about half the shoring load, using high-early cement for the columns, and transferring about half the load on the bars.

Fig. 21, 22 and 23 show the shoring and replacement details. The lintels along the north wall were all very seriously damaged. The wall columns were badly spalled and the column capital was so badly cracked that they had to be removed completely between column rows 4 and 14, inclusive. The expense of shoring the wall columns with the upper stories intact and the projection at the first and second floors would have been prohibitive. It was decided to leave those sections in place. The repairs to the wall columns were made as shown in Fig. 24 by cutting the columns back to the spiral, and pouring new columns and brackets.

Details of repairs to other columns supporting the seventh floor appear in Fig. 26. The north wall lintels were cut back so that all damaged concrete was removed. They were then reinforced, and gunited to their original size, except for lintel L-708 which had to be made two inches wider. To take most of the floor load off these lintels, a stiff wide continuous beam was poured. Details are shown in Fig. 25.

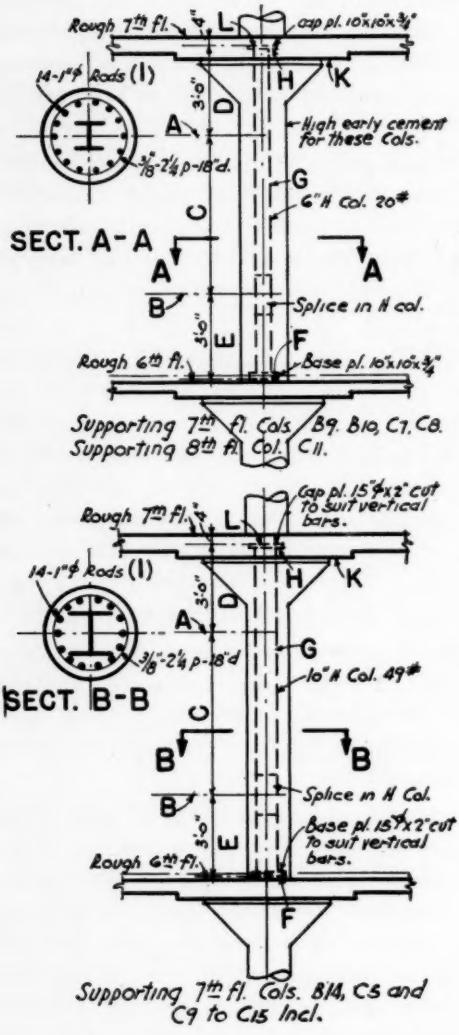


FIG. 23—COLUMN REPLACEMENT DETAILS

Operations

A & B—Cut spiral, and vertical steel on planes *A* & *B*.

C—Remove column section *C* complete.

D & E—Remove spiral, and cut out sections *D* & *E* leaving vertical steel intact.

F—Bed under, and level base plate *F* with waterplug, and wait ½ hour.

G—Set H column after collapsing new spiral around it.

H—Pack above plate *H* with waterplug.

I—Place vertical steel, and spacers and fasten spiral at spacer notches.

K—Pour columns to underside of drop panel.

L—Pack space between column and outside of spiral with high early concrete, and keep damp for 2 days.

M—Remove shores.

The east end of the floor was poured to the construction joint between columns 12 and 13. Then the next two bays were poured. In the meantime some cumbersome machinery was raised and supported from the slab above. After that, the west end was poured to the construction joint between columns 5 and 6 to provide room for equipment that was stacked up so closely that work on them was held up. The balance of the floor was poured on July 1.

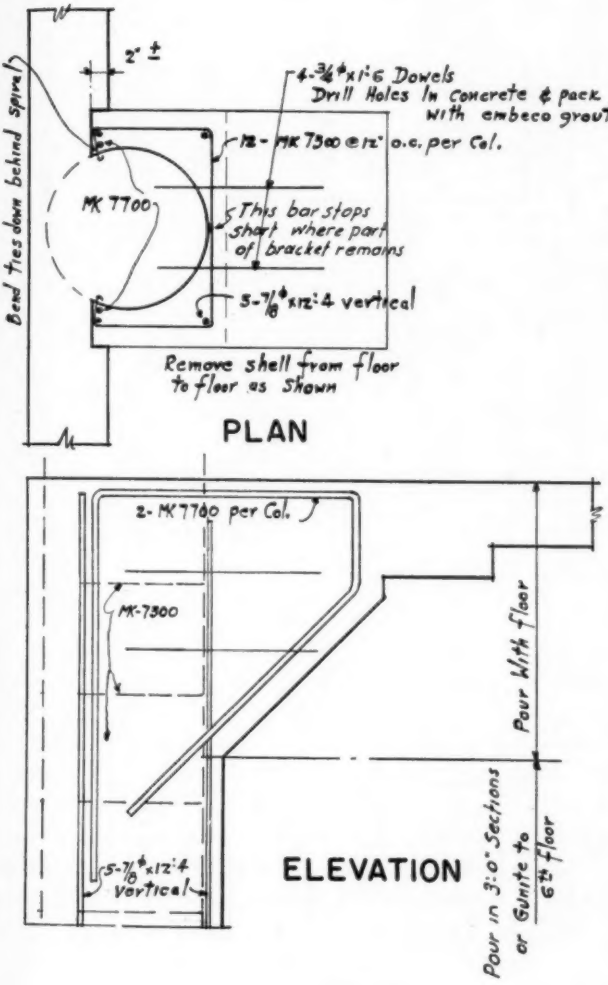

2" ±
4-3/4"ø x 1'-6 Dowels
Drill Holes In Concrete & pack with embeco grout.
12 - MK 7300 @ 12" o.c. per Col.
MK 7700
This bar stops short where part of bracket remains
5-7/8"ø x 12'-4 vertical
Bond ties down behind spiral
Remove shell from floor to floor as Shown
PLAN
2- MK 7700 per Col.
MK-7300
5-7/8"ø x 12'-4 vertical
ELEVATION
Pour with floor
Pour in 3'-0" Sections or Gunite to 6th floor

FIG. 24—REPAIRS TO WALL COLUMNS, SUPPORTING SEVENTH FLOOR, NORTH WALL (E4 to E14)

Eighth Floor

In the meantime, a check-up of our guniting costs revealed that, where no other considerations were involved, it was more economical to remove and replace areas than to repair them. Accordingly, the cutting lines on this floor were extended over what would have been done in the first place. They were left as shown on Fig. 8. Some repair details are shown in Fig. 27.

Before cutting was started on the eighth floor, the seventh floor below was covered with 2 in. plank. Extra layers of plank were placed

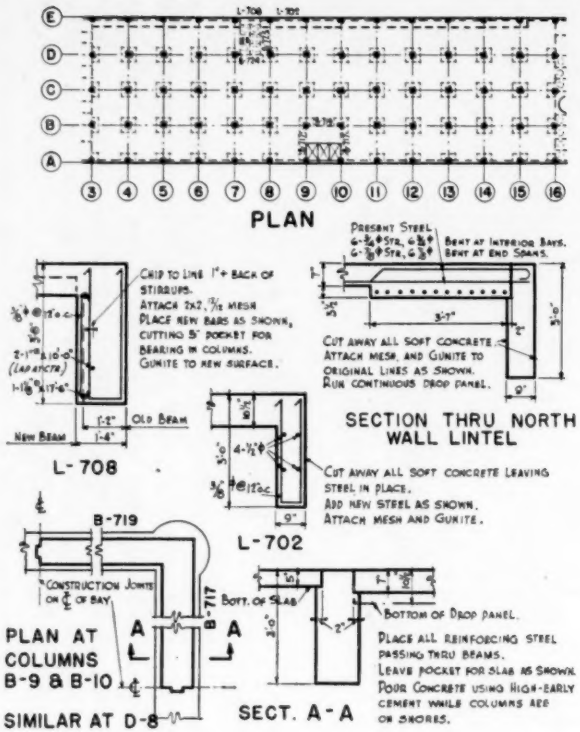

PLAN
L-708
SECTION THRU NORTH WALL LINTEL
L-702
PLAN AT COLUMNS B-9 & B-10
SIMILAR AT D-8
SECT. A-A
B-719
B-717
CHIP TO LINE 1" BACK OF STIRRUPS. ATTACH 2x2-13/12 MESH PLACE NEW BARS AS SHOWN, CUTTING 5" POCKET FOR BEARING IN COLUMNS. GUNITE TO NEW SURFACE.
PRESENT STEEL BENT AT INTERIOR BAYS. BENT AT END SPANS.
CUT AWAY ALL SOFT CONCRETE. ATTACH MESH, AND GUNITE TO ORIGINAL LINES AS SHOWN. RUN CONTINUOUS DROP PANEL.
CUT AWAY ALL SOFT CONCRETE LEAVING STEEL IN PLACE. ADD NEW STEEL AS SHOWN. ATTACH MESH AND GUNITE.
BOTTOM OF DROP PANEL.
PLACE ALL REINFORCING STEEL PASSING THRU BEAMS. LEAVE POCKET FOR SLAB AS SHOWN. POUR CONCRETE USING HIGH-EARLY CEMENT WHILE COLUMNS ARE ON SHORES.
CONSTRUCTION JOINTS ON ℄ OF BAY
BOTT. OF SLAB
OLD BEAM
NEW BEAM

FIG. 25—SEVENTH FLOOR REPAIRS, SEE TEXT PAGE 239

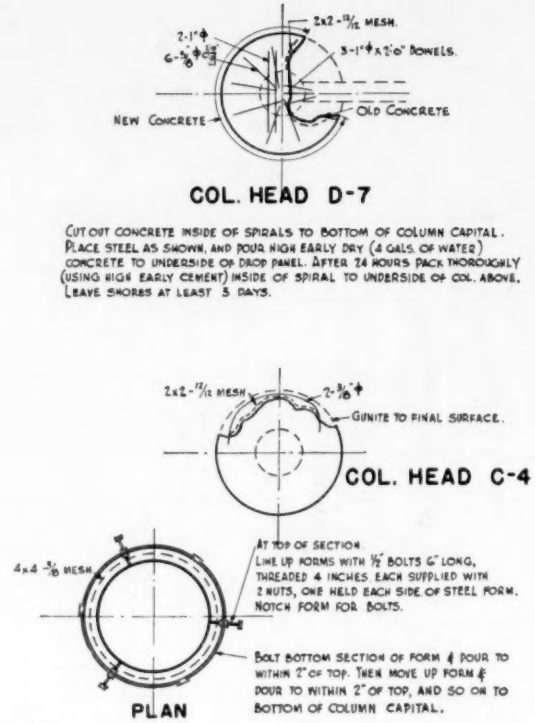

COL. HEAD D-7
CUT OUT CONCRETE INSIDE OF SPIRALS TO BOTTOM OF COLUMN CAPITAL. PLACE STEEL AS SHOWN, AND POUR HIGH EARLY DRY (4 GALS. OF WATER) CONCRETE TO UNDERSIDE OF DROP PANEL. AFTER 24 HOURS PACK THOROUGHLY (USING HIGH EARLY CEMENT) INSIDE OF SPIRAL TO UNDERSIDE OF COL. ABOVE. LEAVE SHORES AT LEAST 3 DAYS.
NEW CONCRETE
OLD CONCRETE
COL. HEAD C-4
GUNITE TO FINAL SURFACE.
AT TOP OF SECTION LINE UP FORMS WITH ½" BOLTS 6" LONG, THREADED 4 INCHES EACH SUPPLIED WITH 2 NUTS, ONE HELD EACH SIDE OF STEEL FORM. NOTCH FORM FOR BOLTS.
BOLT BOTTOM SECTION OF FORM & POUR TO WITHIN 2" OF TOP. THEN MOVE UP FORM & POUR TO WITHIN 2" OF TOP, AND SO ON TO BOTTOM OF COLUMN CAPITAL.
PLAN

FIG. 26—REPAIR OF COLUMNS SUPPORTING SEVENTH FLOOR; AT BOTTOM, METHOD OF POURING COLUMN SHELL

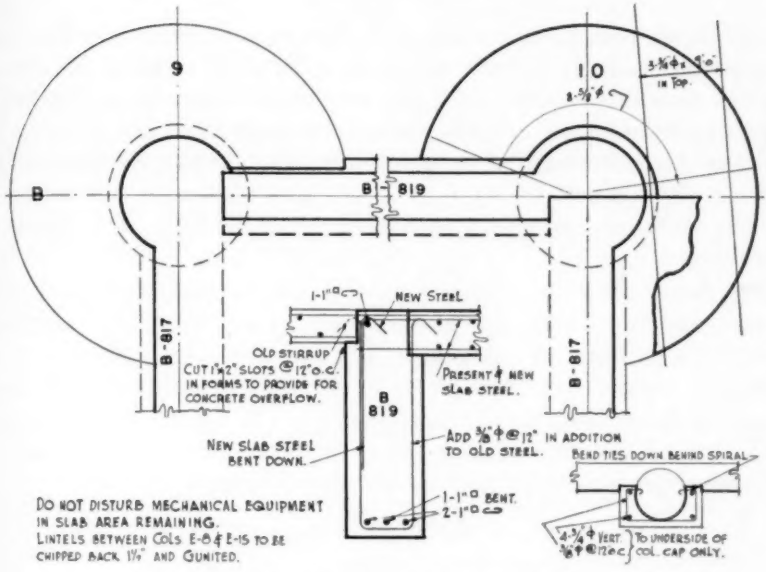

9
10
B
B 819
B-817
B-817
1-1" □
NEW STEEL
OLD STIRRUP
CUT 1"x2" SLOTS @ 12" O.C. IN FORMS TO PROVIDE FOR CONCRETE OVERFLOW.
PRESENT # NEW SLAB STEEL.
B 819
NEW SLAB STEEL BENT DOWN.
ADD ⅝" # @ 12" IN ADDITION TO OLD STEEL.
BEND TIES DOWN BEHIND SPIRAL
1-1" □ BENT
2-1" □
DO NOT DISTURB MECHANICAL EQUIPMENT IN SLAB AREA REMAINING.
LINTELS BETWEEN COLS E-8 & E-15 TO BE CHIPPED BACK 1½" AND GUNITED.
IN TOP
VERT. TO UNDERSIDE OF COL. CAP ONLY.

FIG. 27—EIGHTH FLOOR REPAIRS

under the horses, the legs of which were raised two feet. All this was done to reduce the effects of the dropping of broken concrete on a floor that was only one week old. The centers of the bays were cut out first, so as to give the seventh floor as much chance as possible to harden before the eighth floor was completely cut out, since the tension bracing on this floor was not carried to the outside walls. Otherwise, the procedure was more or less as given for the other floors. Pouring was done in three sections: the first, to the center line between bays 12 and 13; the second, between rows 10 and 11, to the center line between column rows B and C, and then to the center line of bays 9 and 10, to the service shaft.

Near the service shaft, it was originally decided to put the construction joint through the center of the slab. As that particular area was not seriously damaged, and its removal would inconvenience other trades, it was left in place, and the cutting done as shown in Fig. 27. The bent slab bars (already on the job), were bent down into the new beams as shown.

Sixth Floor

The cutting lines for the sixth floor are shown in Fig. 7. It will be noted that there was a good deal of bad spalling along column row 10, that was not cut out, but was repaired by guniting. This left a

beam tie on the center line of the building, and avoided cutting too many seats into the columns while they were not shored. The reinforcing rods in the slab, running north and south were in the lower layer, and the upper rods were not so seriously affected. Because of the stiff beam running along column row 10, it was felt that those lower rods were not taking much load anyway, and rather than cut all the additional area to a satisfactory construction joint, this was left in place.

The beam between column rows E-9 and E-10 was cut back five inches, and two additional one-inch rods added, one in the top, and one in the bottom. Three-eighth inch additional loop stirrups were anchored around these bars at 12 in. on center, and the beam then prepared for pouring in the regular north wall beams were repaired with gunite. Columns removed were shored down to the fourth floor only, as the stub column method of repair was used.

REPAIR TESTS

Columns

To check the safety of the repaired column cores, we decided to test a full size section of a borderline column. Allowing for three-foot dowels left at the bottom, and three-foot dowels at the top of the columns, and two feet for concrete cores and steel specimens, and about three feet for the disturbed concrete and steel at the points where the cutting took place, all that was left for testing was a specimen 28 in. long. Column C-15 supporting the seventh floor, near the extreme end of the severe fire, and the first column to be removed in accordance with the construction schedule, was chosen for the test.

Three cores were drilled from the removed column; one near the outside of the core showed a compressive strength of 1947 p.s.i.; one from the center of the column tested 2554 p.s.i.; and a check core near the outside tested 1739 p.s.i. This last specimen, however, had a ratio of height to diameter of three instead of the usual two. The vertical steel sample showed an elastic limit of 45,580 and the spiral, 52,490 p.s.i.

Cores taken from column C-14 at the same elevation as those taken from column C-15 showed a compressive strength of 1722 lb. near the outside shell and 2391 lb. at the center of the core.

This would give us a weighted average compressive strength at the elevation of the drilled cores of 2100 lb. Since the full size specimen was 2 ft.-6 in. below that plane, and practically all of its protective shell was not removed until after the fire, and since the maximum temperature was near the top of the column, it is reasonable

to assume that the concrete in the test specimen had an average compressive strength of 2400 lb. and the vertical reinforcing steel, an elastic limit of 52,000 lb. The spiral sample was taken at the end of the full size specimen and its elastic limit can be assumed as given. The wet curing of the test column for 18 days increased the concrete strength to about 2800 lb.

Cylinders taken from the concrete as it originally went into the structure tested between 3459 and 4298 lb. at 28 days. The ⅜ in. spiral had an elastic limit of 46,500 lb., and the 1 in. vertical bars, 48,500 lb.

The following is from the report of F. E. Richart, who made the test:

The section of column was tested in a 3,000,000 lb. capacity Southwark hydraulic testing machine * * * and load was applied slowly, in steps of 50,000 lb. A measurement of the over-all shortening of the column was measured by means of a micrometer dial gauge, reading to 0.001 in. In a load-deformation curve (See Fig. 28) it will be noted that the curve is fairly straight up to a load of 1,200,000 pounds, and then begins to show a more rapid rate of shortening with increase in load. The slight divergence from a straight line at loads below 1,200,000 lb. is probably due to lack of perfect bearing at the ends of some of the reinforcing bars. The strain at the load of 1,200,000 lb. is evidently well beyond the yield point of the bars, even allowing for some imperfection in bearing at the ends of some bars.

The maximum load carried was 1,667,000 lb. The column showed considerable spalling and cracking at this stage and had shortened about 0.2 in. As the machine head was run down further the column shortened rapidly and the load fell off to about 1,400,000 lb., when several bars buckled outward and the reinforcing spiral broke. It should be noted that at two places the spirals had been displaced considerably during construction of the column with the result that the pitch of the spiral was increased from the normal 2¼ in. to 3½ and 4 in. respectively. The break occurred near the place where the spiral spacing was 4 in.

From the test results, an analysis of the contribution of the various elements of the column was made, using the method given in Bulletin 267 of the University of Illinois Engineering Experiment Station.

The strength, P, of the column may be estimated from the equation

$$P = Cf'_c\,(1 - p)\,A_c + f_y A_s + Kp'f'_s A_c$$

Where f'_c = strength of concrete control cylinders
A_c = area of concrete core
f_y = yield point strength of vertical bars
A_s = area of vertical bars
p' = ratio of volume of spiral steel to A_c
C = a constant, taken as 1.0 for this short column
K = a constant, usually having a value of 2.0
f'_s = useful limit stress of spiral reinforcement (assumed as the stress at a unit deformation of .005)

$$p = \frac{A_s}{A_c}$$

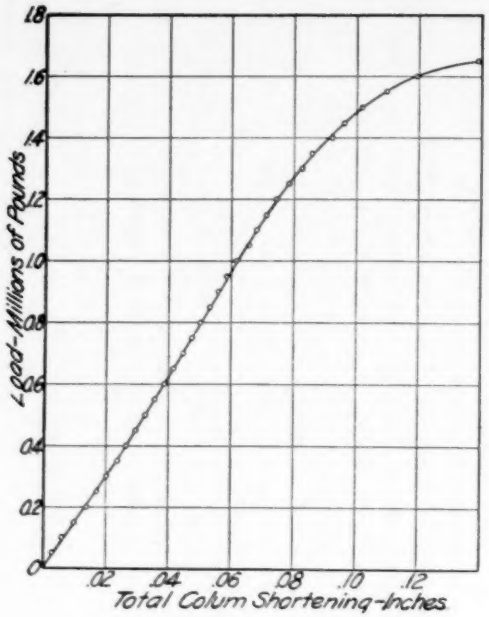

Load-Millions of Pounds
Total Colum Shortening-Inches
0
.02
.04
.06
.08
.10
.12
.14
.16
.18

FIG. 28

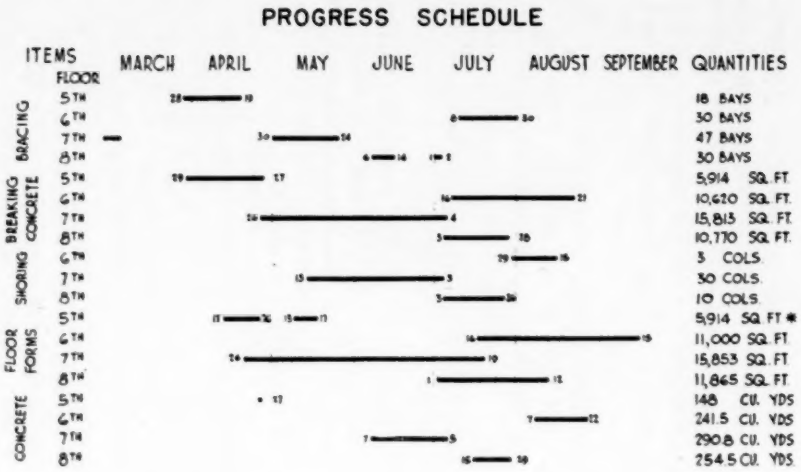

PROGRESS SCHEDULE
ITEMS
FLOOR
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
QUANTITIES
BRACING
BREAKING CONCRETE
SHORING
FLOOR FORMS
CONCRETE
5TH
6TH
7TH
8TH
18 BAYS
30 BAYS
47 BAYS
30 BAYS
5,914 SQ. FT.
10,620 SQ. FT.
15,813 SQ. FT.
10,770 SQ. FT.
3 COLS.
30 COLS.
10 COLS.
5,914 SQ. FT.
11,000 SQ. FT.
15,853 SQ. FT.
11,865 SQ. FT.
148 CU. YDS
241.5 CU. YDS
290.8 CU. YDS
254.5 CU. YDS

FIG. 29

$Cf'_c\ (1 - p)\ (A_c)$ = 1 x 2,800 x 243.5	=	682,000
$f_y A_s$ = 11 x 52,000	=	572,000
$Kp'f'_s A_c = K(.0109)\ (52{,}500)\ (254.5)$	=	413,000
or K = 2.83		1,667,000

The columns most seriously affected by the fire, because of their location and length of exposure, as evidenced by the spalling of the shell, low strength of cores taken near capitals, calcining of concrete inside of spirals, and crumbling when chipper was applied, were removed completely. Others had the upper 2 ft.-6 in., including the capitals, replaced. And in all cases, columns remaining were chipped back of spiral to remove damaged material and replaced with packed or gunited concrete. Additional reinforcing was added wherever the shell was replaced.

Going back to the test, a column might conceivably have been left in place and repaired where only a small section was more or less seriously affected. In those cases a new capital was inserted, the old shell and the affected material inside the spiral completely removed, and replaced, and additional steel added. For most of the shell to remain, the temperature could hardly have been 1000°F. Assuming a small section to be exposed say to 1400°F., which is about the maximum variation to be expected, and using the same formula, allowing for the new steel, and effect on the old

Effect of vertical steel

New steel 1.0 (64,000)	=	64,000
Exposed steel 4.74(42,000)	=	199,100
Protected steel 6.32(52,000)	=	328,600
		591,700 lb.

Effect of concrete

C now is .85 because we are considering a column 12 ft. 6 in. high,

243.5 x 3,000 x .85	=	620,900 lb.

Effect of spiral

K assumed as 2.0 (reasonable minimum)

Old spiral .0109 x 254.5 x 42,000 x 2.0	=	233,000
New mesh .0016 x 254.5 x 64,000 x 2.0	=	52,000
		285,000 lb.

P = 1,497,600
Design Load = 394,000

That is, assuming an extreme condition, we still would have a factor safety of 3.8. Actually, any such columns would be removed, since the reduced elastic limit of the steel, would be accompanied by a soft concrete having an average compression strength of under 2,000 p.s.i. on the entire cross section, and ordinary economy would make it desirable to replace the entire column, rather than repair it.

Floors

To check the efficiency of the floor repairs a load test was made on a repaired section. Three-quarters of the bay bounded by column rows B, C, 11, and 12 of the fifth floor was replaced. A strip two to four feet wide along the edge of the remaining section was, because of its condition, chipped back, and repaired by gunite. Several cracks had been left, without any effort at repair. All in all, it seemed that if this section withstood a loading test, the repairs would be considered successful. Accordingly, this bay was selected for the test.

Bearing in mind that only one bay was loaded, the results, (in Fig. 16), exceeded our expectations. We were a little disappointed that the recovery, after the load had been removed for 12 hours, was only 85 per cent.

After this test a check was made on the same floor, the bay between column rows 6, 7, C, and D—the most seriously cracked of those left unrepaired. Several other bays were tested where a test seemed desirable. To save time and expense, unless something unforseen should develop, the tests were to be carried only to a point where four layers of pig iron were placed on the bay (approximately 250 lb. per sq. ft.). The load was left on for 12 hours and then removed. As no cracks developed at any of those points and the deflections and recovery were satisfactory, these light tests seemed sufficient.

UNIT COSTS

In the following repair cost data no insurance, job overhead or contractor's office overhead is included:

Bracing (per average bay)
Materials *$94.00
Labor (skilled, 14.75 hrs. @ 1.30 = 19.17; common, 11.5 hrs. @ .65 = $7.48) 26.65
Total Cost per average bay $120.65

Breaking Concrete (per square foot of floor area)
Scaffold
Material ..... .005
Skilled labor ..... .005
Common labor ..... .005
say 0.01
Breaking concrete
Skilled labor .015 hrs. @ 1.25 = .02
Common labor .18 hrs. @ .85 = .15
0.17
Removing debris
Skilled labor .007 hrs. @ 1.33 = .01
Common lab. .062 hrs. @ .65 = .04
0.05
Total cost per square foot of floor area ..... = $0.23

Trimming Edge of Slab, or Cutting Bearing Ledge (per lin. ft.)
Common labor .75 hrs. @ .85 ..... $0.64

Cutting Key at Construction Joint (per lin. ft.)
Common labor .28 hrs. @ .85 ..... $0.24

*Less Salvage Value (33⅓ per cent).

Drilling Holes for Dowels (per dowel—9 in. deep)
Common labor .21 hrs. @ .85 ..... $0.18

Complete Column Replacement (per Column)
a) *Shoring*
Timber and Reusable Steel *$166.00
Erection
Skilled labor 165 hrs. @ 1.30 ..... = $214.50
Common labor 75 hrs. @ .65 ..... = 48.75
263.25
Removal of Shoring
Skilled labor 1.75 hrs. @ $1.30 ..... = 2.28
Common labor 21.5 hrs. @ .65 ..... = 13.97
16.25
b) Structural steel
Material $47.67
Skilled labor 6.2 hrs. @ 1.50 ..... 9.30
c) Reinforcing steel
Material 9.00
Skilled labor 4.3 hrs. @ 1.50 ..... 6.45
d) Cutting and removing concrete
Skilled labor 2.2 hrs. @ 1.50 ..... 3.30
Unskilled labor 36 hrs. @ .65 ..... 23.40
26.70

e) Concrete
Material 9.00
Common labor 4.5 hrs. @ .65 .... 2.93
f) Forms
Material rental 4.50
Leased labor 7 hrs. @ 1.95 ..... 13.65
g) Bearing plate
Material (included in structural steel)
Material for packing 3.00
Setting plates 2 hrs. @ 1.25 ..... 2.50
Total Cost per column replacement . $580.20

*Less Salvage Value (33⅓ per cent).

Column Capital Replacement (per Column)
a) *Shoring*
Timber and reusable steel *$143.00
Erection
Skilled labor 131 hrs.
@ 1.30 ..... = 170.30
Common labor 61 hrs.
@ .65 ..... = 39.65
209.95
Removal of Shoring
Skilled labor 1.5 hrs.
@ 1.30 ..... = 1.95
Common labor 17.5 hrs.
@ .65 ..... 11.38
13.33
b) Structural steel
Material 5.40
Skilled labor 2.15 hrs. @ 1.50 .... 3.23
c) Reinforcing steel
Material ....
Skilled labor 1 hr. @ 1.50 ..... 1.50
d) Cutting concrete and removal
Common labor 20.5 hrs. @ .65 ... 13.33
e) Concrete
Material 4.00
Common labor 2 hrs. @ .65 ..... 1.30
f) Forms
Material lease 4.50
Leased labor 2.9 hrs. @ 1.95 ..... 5.66
g) Bearing plates
Material (included in structural steel)
Material for packing 1.00
Setting plates 1 hr. @ 1.25 ..... 1.25
Total cost per column capital $407.45

*Less Salvage Value (33⅓ per cent)

Gunite Costs (per square foot—2 in. thick)
Chipping .31 hrs. @ .85 ..... $0.26
Mesh and bolts
Material 0.06
Erection 0.07
Form strips for
a) Square columns
Material 0.14
Labor .24 hrs. @ 1.30 .. 0.31
0.45
b) Ceilings
Material 0.03
Labor .06 hrs. @ 1.30 .. 0.08
0.11
c) Beams
Material 0.20
Labor .37 hrs. @ 1.30 .. 0.48
0.68
Gunite
Material 0.06
Leased labor placing including rental and operation of machine 0.43
Finishers for
a) Square columns
Labor .09 hrs. @ 1.25 ..... 0.11
b) Round columns
Labor .20 hrs. @ 1.25 ..... 0.25
c) Ceilings
Labor .18 hrs. @ 1.25 ..... 0.22
d) Beams
Labor .23 hrs. @ 1.25 ..... 0.29
Total cost for 2 inch depth of Gunite
a) Square columns ..... $1.44 per sq. ft.
b) Round columns ..... 1.13 per sq. ft.
c) Ceilings ..... 1.21 per sq. ft.
d) Beams ..... 1.85 per sq. ft.

Pouring 2-in. Shell on Round Core
Chipping (.31 hrs. @ .85) $0.26
Mesh and bolts $0.06
Erection 0.03
Forms 0.09
Lease of Material 0.11
Placing and stripping forms
Leased labor .09 hrs.
@ 1.95 ..... 0.18
0.29
Concrete
Material 0.03
Labor .11 hrs. @ .65 ..... 0.07
0.10
.056 hrs. @ 1.25 ..... 0.07
Total cost per square foot $0.81

Concrete (per yard)
Material (delivered) $4.85
Placing (including runways)
Common labor .97 hrs. @ .65 .... 0.63
Skilled labor .18 hrs. @ 1.30 ..... 0.23
Total cost per yard $5.71

Forms per Square Foot Contact Area
Material ..... * $0.16
Labor placing and stripping slabs
Skilled labor .102 hrs.
@ 1.30 ..... = .133
Common labor .069 hrs.
@ .65 ..... = .045
0.18
Beams and drop panels
Total labor cost 0.42
Edging
Total labor cost 0.33
Bulkhead
Total labor cost 0.54
*Less 25 per cent Recoverable Value.

ACCIDENT RECORD

With seven swinging scaffolds in continuous operation and one or more floors open at all times, there were but two lost time accidents (sprained backs due to shoring operations.)

One other accident occurred on the ground, when a laborer operating a concrete mixer for the floor finish, lost his foot in trying to clean the mixer. This man was not, however, on the general contractor's payroll.

ACKNOWLEDGMENTS

The task of reconstruction was assigned by General Mills, Inc. to Henry S. Crosby, Chief Engineer of the company, Fritz H. Nelson of Cereal Construction Co., and the writer.

Final decisions as to scope and methods of repairs were made by Mr. Crosby. His untiring efforts at the start of the project, unfailing encouragement, and quick arrival at conclusions throughout the operation, were of utmost importance. Coordination and general supervision were in the capable hands of Mr. Nelson.

The instructions of General Mills to keep a complete record of the operations for a full report makes this paper possible. Outside the organization entrusted with the project, we are especially grateful to Nolan D. Mitchell, of the Bureau of Standards, for many helpful suggestions in the preparation of the report. Suggestions have also been received from J. C. Pearson, Lehigh Portland Cement Co., A. J. Boase and F. R. McMillan of the Portland Cement Association.

The repair contract was executed by the Hydro Construction Co. The concrete skeleton was originally erected by the Charles H. Wing Co. The Cereal Engineering Co., Minneapolis, Minn., acted as owner's representative.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June 1941.*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JUNE, 1941*

Discussion of a paper by J. Fruchtbaum:

Fire Damage to General Mills Building and Its Repair*

AT THE CONVENTION AND BY S. H. INGBERG AND THE AUTHOR

CONVENTION DISCUSSION

Nolan D. Mitchell† (in response to a question from the convention): Of course the strength of concrete after being subjected to high temperatures is a function of the aggregate as well as the temperature. Some aggregates will result in very early failures, failures at a comparatively low temperature, while others will hold up very much longer, so the answer cannot be a categorical one. But as for the temperatures Mr. Fruchtbaum is probably correct. The temperature of the atmosphere did, in some locations, reach as high as two thousand degrees but of course we could not expect the heat to penetrate the concrete so very rapidly, and while some small particles might have reached a temperature approaching that of the atmosphere, it is not likely that there was such temperature to any great depth in the concrete. This matter of failure of concrete at high temperatures is one we should know a great deal more about than we do at present. I, for one, hope that we can have answers to some of these questions that are much clearer as to the behavior of the aggregates as well as to the behavior of the cement under fire exposures.

Mr. Fruchtbaum (in response to an inquiry from the floor): About 55 tons of reinforcing bars were actually replaced and about 15 tons were added in those locations where it was felt the structure should be strengthened even though not completely removed. To check settlement, two men were employed checking elevations and key measurements throughout the shoring operation. We found no measurable settlement of the columns repaired by the stub method.

*JOURNAL, Amer. Concrete Inst., Jan. 1941; *Proceedings* V. 37, p. 201; presented 37th Annual Convention, Washington, D. C., Feb. 19-20, 1941.

†National Bureau of Standards, Washington, D. C.

On one of the first columns, a settlement of .02 ft. was recorded, but that reading was subject to question.

BY S. H. INGBERG*

The estimation of temperatures attained in building fires has been based on fusion effects on materials of known melting points. The fused glass cited by the author can be taken as indicating minimum temperatures of 1300 to 1400°F. and the drooped bronze sprinkler deflector, temperatures of about 1700°F. The calcination of the concrete, which in this case would be largely of the portland cement binder, takes place over a wide temperature range and is as much a function of time as of temperature. This represents the extent of the evidence presented in support of the estimations of temperature and duration made in the paper. The effect of temperature and its duration on the properties of the reinforcing steel was not correlated with the probable temperature and duration of the fire in the different portions of the building, and on account of the many factors involved, there apparently exists little possibility of thus obtaining even approximate indications.

The type of contents involved were favorable for the development of high temperatures, but it is doubtful whether any as high as 1700 to 2000°F. would persist in any one location or even on a given floor for the periods indicated in the paper, since after the first active combustion the layer of ash formed and collapse of burning material into a mass materially decrease the intensity of the fire. The air supply also has an important bearing thereon, and only when this is ample but not greatly in excess of requirements for combustion can temperatures as high as 2000°F. be maintained. While the amount of combustibles involved might under the most favorable conditions give fires equivalent in severity to 8 or even 12 hours of the standard furnace test, the largest part of the area under the time-temperature curve, on which the equivalence is based, would probably have temperature ordinates much below the maximum. Nor are such long durations of high temperature needed to account for the structural damage incurred since severe disruption of concrete made with the general type of aggregates concerned has been obtained in fire tests of less than one hour duration and maximum indicated temperature of the exposing fire below 1700°F.

As concerns the immediate cause of such fire effects, while the high expansion of the silica aggregates is without doubt a contributing factor, it appears that restraint such as that afforded by the surround-

*National Bureau of Standards, Washington, D. C.

ing construction must be present if damage of any consequence is to occur. In recent fire tests with restrained and unrestrained slabs of concrete made with aggregates consisting almost wholly of quartz and chert sand and pebbles, no spalling or other disruptive effects were obtained with unrestrained slabs, while openings were formed in 4-in. restrained slabs before the end of the first test hour.

AUTHOR'S CLOSURE

With reference to Mr. Ingberg's discussion, it is to be regretted that he failed to include charts which he has prepared showing the relation between building contents and temperatures generated. Some of these were published in the Quarterly of the National Fire Protection Association for July, 1928. These tests were carried out under more or less ideal conditions, while in actual fires the temperature and duration will be affected by the available air supply, lack of uniformity in storage of materials, and location of beams, intermediate walls and openings. These charts will therefore provide an upper limit for temperatures but low values for duration.

We arrived at our temperatures primarily by watching the fire debris. Certain of the evidence pointed to very definite minimum temperatures. Starting with these base values, and comparing samples of reinforcing steel taken throughout the job, we arrived at the temperatures quoted in the report. Our reinforcing steel tests as shown in Fig. 13 A, B, C, and D make a good basis of comparison as all tests were carried out under identical conditions.

Unfortunately the results of the tests of the samples taken from the structure are rather confusing. As far as the elongation tests were concerned, we could not use them at all. We did however get somewhere with our elastic and ultimate limit tests by making allowances for special conditions such as nearness to a sprinkler head, amount of concrete cover left, duration of exposure to fire, and location.

For our own use we plotted curves showing the relation between change of strength in steel and loss of strength in concrete. Too many variable factors made the curves almost useless. We did, however, decide on the general shape of the curves and plotted the points. We were as a general rule able to find some logical reason for some points or group of points falling off the curve. The main function of these charts was seen as a check on the tests of the materials.

I believe Mr. Ingberg refers to discussions in the preliminary stages of the job, as we steered clear of giving definite durations of the high temperatures indicated, just as we steered clear of drawing any definite conclusions. We felt that our data did not justify them.

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JOURNAL
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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY, 1941*

Problems Presented by the Lake Washington Floating Bridge*

By Charles E. Andrew†

SYNOPSIS

The largest pontoon bridge in the world and the first to be built of reinforced concrete presented numerous problems of design and construction, those resulting from the use of concrete being of course original. Some of these are presented in the following paper.

Why a pontoon bridge was selected; why it was built of reinforced concrete; the essential features of the design; how the concrete was made watertight; what results were achieved in this respect; how the contractors built the 25 pontoon sections; how they were assembled, interconnected, anchored; what provision is made for the passage of large ships; what are the bridge's notable features in service—all of these questions and topics are the material with which this paper deals.

INTRODUCTORY—THE PROBLEM

The Lake Washington Floating Bridge at Seattle, Washington is itself the solution of a difficult problem of highway relocation of which the conditions are briefly these:

North and south across the State of Washington from Oregon to British Columbia extend the Cascade Mountains, 5,000 – 8,000 ft. high, rugged, precipitous and blanketed with heavy snows in winter. Their westerly slopes are extremely steep and abrupt and there are only a few low passes where highways can cross them. Of these Snoqualmie Pass, 3,004 ft. high and distant 55 miles from Seattle is by far the best. It is used by State Road No. 2, the principal east-west road of the state and the only road across the Cascades which during the winter months can, economically, be kept open and free from snow. In recent years this highway has been relocated and

*Received by the Institute September 20, 1940.
†Principal Engineer, Washington Toll Bridge Authority, Seattle, Washington.

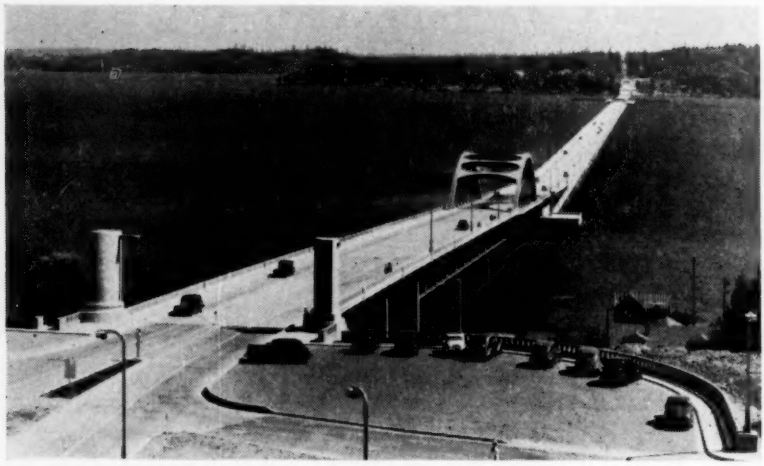

FIG. 1—LAKE WASHINGTON PONTOON BRIDGE, SEATTLE, WASHINGTON

paved for the full distance across the mountains down to the town of North Bend where the lowlands of the Puget Sound basin are reached. The difficult problem of highway relocation referred to was how to route and relocate State Road No. 2 from North Bend to Seattle. (Fig. 2).

Most desirable route of all, of course, would be along the straight line connecting North Bend with the center of Seattle. A close approximation of this straight-line route with excellent grades was indeed available almost to the edge of the city but at that point Lake Washington, a mile and a half wide and 100 to 220 ft. deep, lay squarely athwart it and beyond the lake and within the city was the wide-topped hill or ridge which rises abruptly from the waters of the lake to a height of 250 ft. or more.

Lake Washington forms the eastern boundary of Seattle. It is a large fresh water lake, 20 miles long, 1 to 3 miles wide and very deep for almost its entire length. Seattle lies between it and the main body of Puget Sound, the lake extending several miles both to the north and to the south of the city limits.

Because of its great depth, the lake formed a serious barrier to all automobile traffic between Seattle and the East, such traffic being forced to go around either the north or the south end of the lake or take a poorly and infrequently operated ferry. Connected with Puget Sound by the Lake Washington Ship Canal and locks the lake is subject to ocean-going traffic.

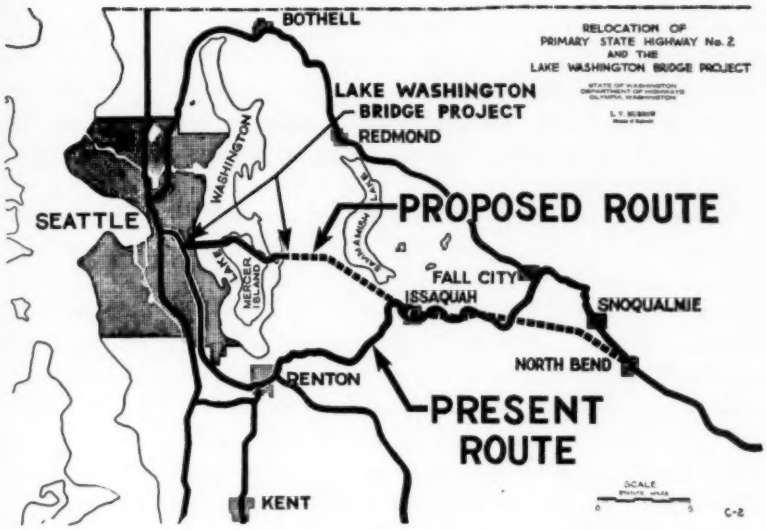

FIG. 2—HOW THE NEW LAKE WASHINGTON PONTOON BRIDGE FITS IN TO THE HIGHWAY ROUTE EAST OF SEATTLE

Several schemes to bridge the lake had been promoted and advanced. Extensive depths of water and soft foundation material made the construction of a bridge on piers certainly impractical, if not impossible, as foundation depths up to nearly 400 feet deep would be required.

Shortly after the world war private interests proposed a pontoon bridge. The scheme was to use obsolete ship hulls as cross-pontoons, bridging between them. Other schemes for building a pontoon bridge followed, but all were abandoned for one reason or another. A bridge further south at Seward Park proposed to be constructed as a trussed superstructure with piers resting on a submerged fill was proposed but could not be financed.

In 1936, Lacey V. Murrow, Director of Highways for the State of Washington, became interested in the project and obtained the approval of a Toll Bridge Law enabling the State to build bridges of this character as self-liquidating projects. Immediately following the enactment of this law, Mr. Murrow directed his Department to make a study of a proper location and type of structure to bridge Lake Washington. During the preliminary discussions of the problem, Homer M. Hadley, Regional Structural Engineer of the Portland Cement Association proposed the development of the route later adopted and crossing the lake with a pontoon bridge made of reinforced concrete pontoons placed end to end. R. B. McMinn, Bridge Engineer of the Bureau of

Public Roads at Portland, Oregon, was asked by Mr. Murrow to report on the feasibility of such a scheme. Mr. McMinn's report was in turn submitted to the writer. In the meantime, Mr. Murrow's Department was investigating other possible types of pontoon structures as well as a conventional bridge on piers including a proper location for the bridge.

A Board of Consultants composed of the writer as Chairman, R. B. McMinn, R. B. Thompson and Rear Admiral (Ret.) Luther E. Gregory, was appointed to act in conjunction with the Department of Highways. A full year was spent in the study of every phase of the problem, the result of these studies being the bridge as it now stands on a direct line between the approximate center of gravity of population in Seattle and Route 10 to eastern Washington.

REINFORCED CONCRETE PONTOONS

Once the decision to build a pontoon bridge was reached, the next problem that arose was of what material the pontoons should be constructed. Obviously they could be constructed of wood, steel, and reinforced concrete. Detailed designs and cost estimates were prepared for pontoon units of these three materials and for their respective necessary anchorage systems. It is obvious that the lighter a floating object is the more readily may it be tossed and put in motion by waves; further, if it is to be held closely to some desired fixed position, then the lighter the object the more taut and rigid must be its anchorage. For a pontoon bridge reinforced concrete with its continuity and freedom from joints and its great mass and dead weight has points of distinct advantage and superiority and inasmuch as the cost estimates also favored it, it was chosen as the material for the construction of the pontoons. The concrete ships built in 1917-18 left no doubt in our minds as to the structural sufficiency of concrete for such a use and a concrete barge, the Hercules, built in Seattle in 1918 and in continuous service thereafter in both salt and fresh water gave convincing local evidence that concrete could be made which would not deteriorate.

A four-lane roadway and sidewalks being wanted led to the development and adoption of a typical pontoon unit whose cross-section is shown in Fig. 3. It is essentially a rectangular box-girder divided by longitudinal and transverse webs into substantially cubical cells whose sides where subject to transverse load are reinforced as two-way slabs. The typical unit has an overall length of 349 ft.-10 in. This is divided into 12 watertight compartments by alternate transverse webs or bulkheads between which the individual cells are interconnected by large openings in the webs. Attention may well be directed to the simplicity of this framing and its formwork: it is free from ribs

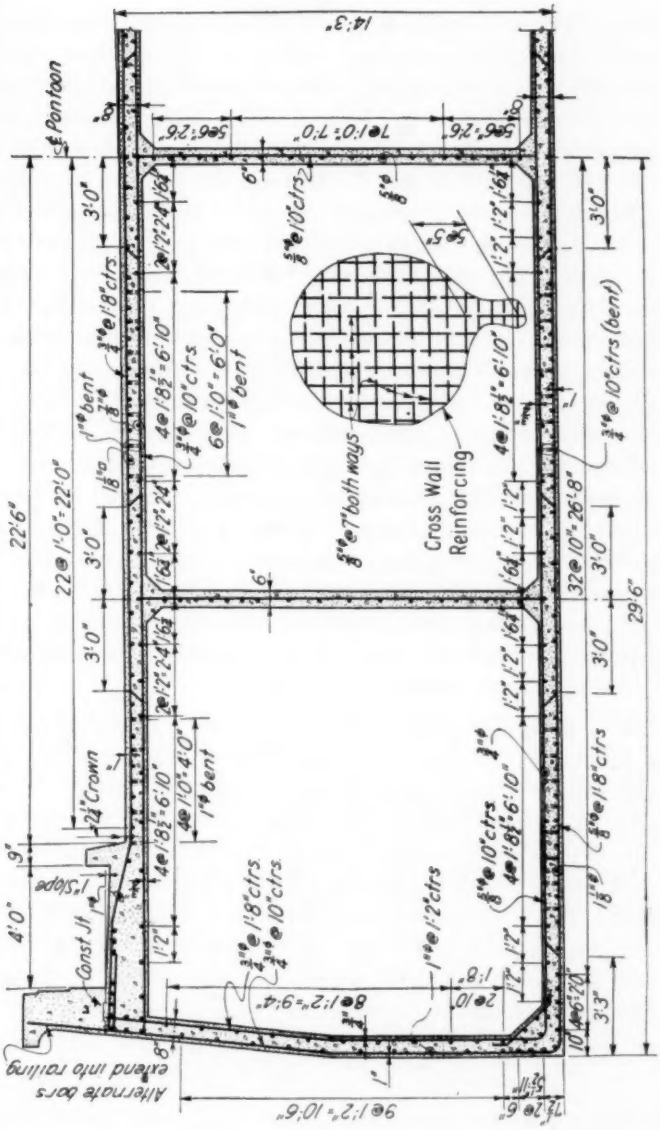

FIG. 3—HALF SECTION, TYPICAL PONTOON

and pilasters, either vertical or horizontal. The outer slabs, top, bottom and sides are 8 in. thick, the inner webs are 6 in. thick. The total thickness of longitudinal web for the entire section is 34 in.

THE CONCRETE

A primary assumption of the design was that the pontoons would be watertight and that leakage would be negligible, such as might

occur being slight and to be removed by occasional pumping. For the realization of this assumption shrinkage in the concrete would have to be held to a minimum. Consequently the cement content of the concrete was held to what was judged to be a minimum quantity for impermeability with the aggregates used and conditions of placement: 1.50 bbl. per cu. yd. Following the concreting of the first unit certain adjustments were made in the mix; thereafter the standard mix consisted of the following proportions per cubic yard batch: 564 lb. of cement; 661 lb. of fine sand; 930 lb. coarse sand to ¼ in.; 816 lb. fine gravel; 983 lb. coarse gravel to 1½ in.; total water content per sack, including water in sand, 5.5 gal.; water cement ratio 0.73; average slump, 6 in. The contractors were very desirous that a fluid admixture, (Vaso), be used and this was permitted, 1 lb. per cu. yd. in the hull but not in the roadway. Apparently the admixture occasioned a slight loss of strength and of weight but did have beneficial effects, both plastically and psychologically. Except for handrails and curbs which were added after the pontoons had been assembled and anchored in the lake, all the concrete was placed in two operations. The first pour and the major one included everything below the bottom of the roadway slab, 1350 cu. yds. It eliminated all construction joints in the lower part of the sections. The second pour completed the roadway slab and sidewalks.

The results obtained with the concrete were highly satisfactory. Watertightness was achieved to a degree exceeding our expectations. When forms were stripped a gravelly spot or patch of honeycomb would occasionally be found. These were cleaned out and the holes were filled either with a damp mortar rammed with air hammers or with Gunite. Curing of interior surfaces with a black membraneous compound (Hunt Process) immediately followed removal of the forms while exterior surfaces were covered with cotton mats which remained in place and were kept wet until the pontoon were floated. Thereafter all the concrete that was immersed has had continuous water curing. Consequently not the slightest tendency to shrink has occurred in the lower portion of the sections. There is the further fact that no friction or restraint exists with water bearing. Above the water line the outer surfaces of the sides and deck have been exposed to sunlight and drying. On the other hand the inner surfaces of these parts are in the cool dark interior of the pontoons. Moreover all parts are very heavily reinforced. Out of this combination of conditions has come the very remarkable result: a complete absence of cracks in the hulls. Only in the curbs which were poured after the individual

pontoons had been assembled and anchored in the lake are fine shrinkage cracks to be found.

As for leakage it has to date been non-existent and there appears to be no logical reason why this condition should change. Except for rainwater which entered through uncovered manholes and that caught by the "drip pans" where the anchor cables enter the sides of the pontoons there has been no water in the interiors. They are bone dry, not only on the sides but on the bottom as well.

THE CONSTRUCTION

The contractors' problem: how to build and launch the pontoons—was solved in a very efficient manner. With 25 pontoons to construct the contractors built a pair of graving docks on Harbor Island on the Puget Sound waterfront of Seattle, constructed the pontoons in them and launched the successive sections without subjecting them to the slightest abnormal stress by simply flooding the docks. Thence they were towed to Lake Washington through the ship locks previously referred to.

The docks are 70 ft. wide, about 15 ft. deep below ground surface, 365 and 400 ft. long respectively. Their sides are of Wakefield sheet piling and their bottoms are 30-in. concrete slabs held down by 13 rows of piles. Five of these rows came directly beneath the sides and longitudinal webs of the pontoons. All piles passed completely through the slabs and were cut off 4 in. above. On these pile heads the wood floors of the docks are framed and supported. Each dock was provided with a lift gate at its open end which was raised at high tide when a pontoon was to be towed out and was closed at the following low tide when the water in dock had sunk to its lowest level. After the gate was closed the remaining water was pumped out and work was commenced on the next unit. The pontoons were constructed directly on the wooden floor of the docks which were shaped to conform with the 3-in. taper in the bottom of the pontoons.

A complete set of outer forms was provided for each graving dock. One set of inner forms was used alternately, first in one dock and then in the other. The forms were made of wood, were carefully and substantially built and were impregnated with form oil. They were handled in panels by the stiff-legged derricks which served the docks. The inner forms were made up in units, one for each cellular space, (Fig. 4) the sides being loosened and tightened by corner wedges. All forms were very carefully aligned and positioned. The outside forms were braced against the sides of the dock and no form bolts or ties passed through the outer shells below water level. The inner forms were first set down on little jacks having movable heads and

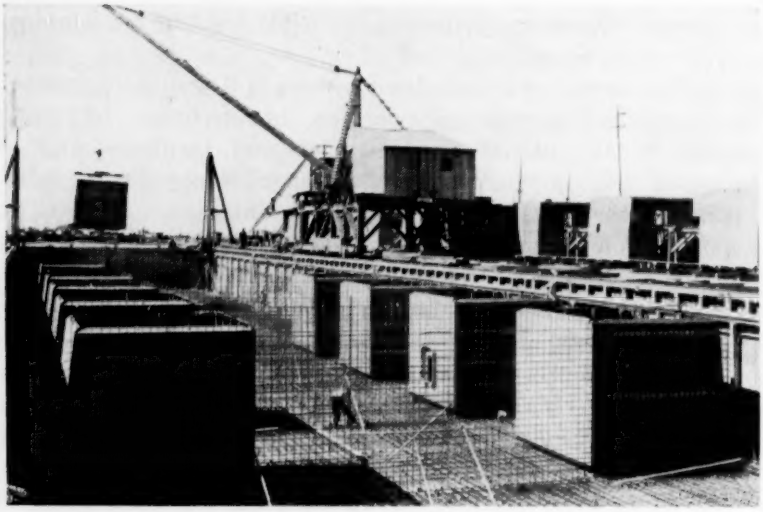

FIG. 4—PLACING REINFORCING STEEL AND CELL FORMS FOR THE FIRST PONTOON, MAY 8, 1939

after being aligned by a transit were lowered onto cast iron chairs which in turn were supported on concrete blocks and were left permanently embedded in the concrete of the bottom slab.

Each dock was spanned by a movable double-decked concreting bridge running on rails for the full length of the dock. On the upper floor concrete was wheeled from an end hopper to a row of elephant trunks equi-spaced along the rear side of the advancing bridge. The lower deck was a working platform from which the lower ends of the elephant trunks were handled and vibrators were operated. The concreting procedure after some initial adjustments consisted, in the case of the main pour, of starting at one end of the pontoon and working slowly and continuously forward until the opposite end was reached. The concrete for the bottom slab was handled by the stiff-legged derricks and was placed about half an hour in advance of the web concrete. The derrick swung buckets of concrete to movable hoppers placed over the individual cells, whence by elephant trunks the concrete of the bottom slab was deposited almost directly in final position. The concrete of the webs was handled via the movable bridge, a caterpillar crane lifting the concrete to the hopper on the upper deck. From this hopper the concrete was wheeled in buggies to the various elephant trunks. Stiffening of the bottom slab concrete during the initial half-hour period was sufficient to prevent the escape of the web concrete which, at the starting end, was built up to the full

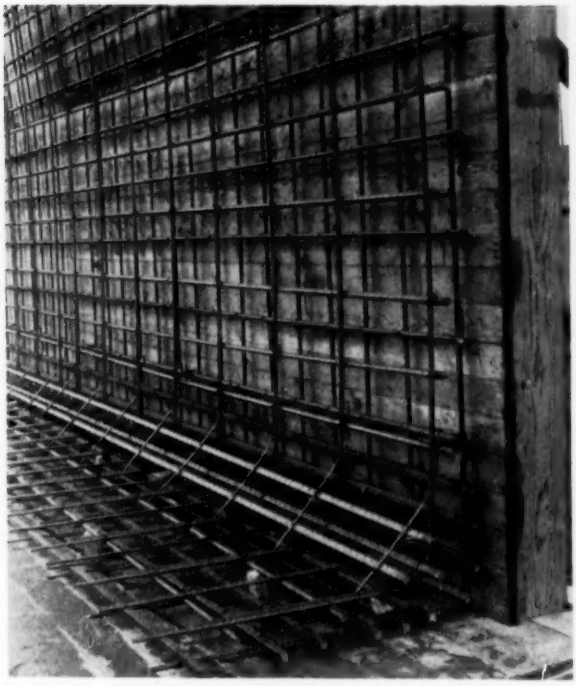

FIG. 5—TYPICAL OUTSIDE WALL REINFORCING STEEL IN PLACE FOR EXPERIMENTAL POUR

Before the contractors attempted any work on the pontoon they built several experimental wall sections of full size and reinforced in the same manner as the pontoons and poured them to obtain some experience on placement. This view shows the reinforcement in one of the experimental walls.

height of the web and from this point was allowed to take its own slope forward. As the bridge advanced the concrete in the webs was vibrated and at the rear of the bridge was brought up to desired level. An average of 70 cu. yds. of concrete was placed per hour.

Forms were removed from the webs after two and one-half days. Forms for the roadway slab were then set in position, reinforcement was placed and the upper deck was poured. Four and one-half days after this pour was completed the dock was flooded and the pontoon floated. In all cases the pontoons came free from the bottom form without any difficulty. The average cycle in each graving dock was 20 working days.

PONTOON CONNECTIONS

For interconnecting the pontoons end to end and maintaining the continuity of the roadway surface a very satisfactory connection

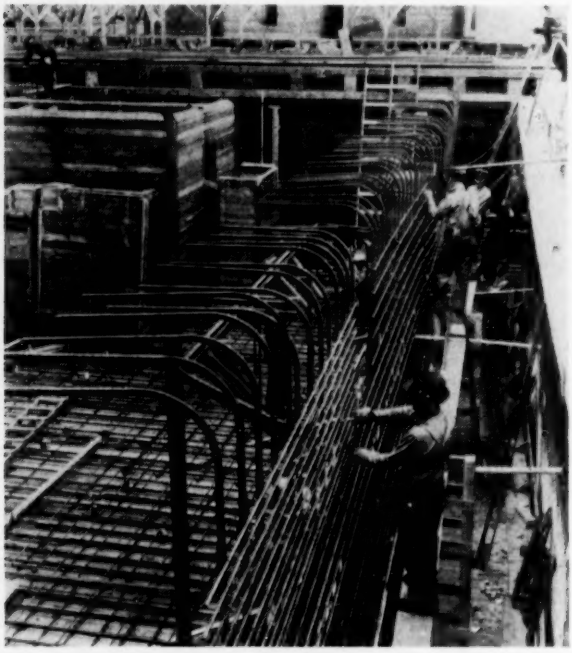

FIG. 6—TYING REINFORCING STEEL FOR END WALL OF FIRST PONTOON, GRAVING DOCK NO. 1

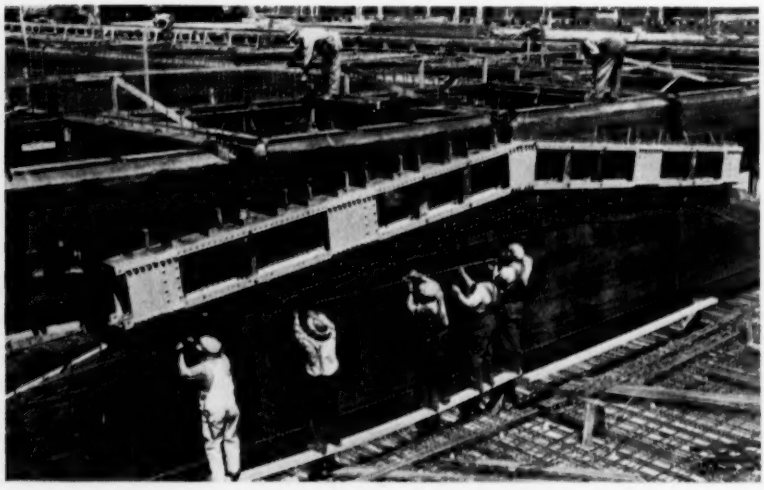

FIG. 7—INSTALLATION OF CABLE ANCHORAGE STEEL

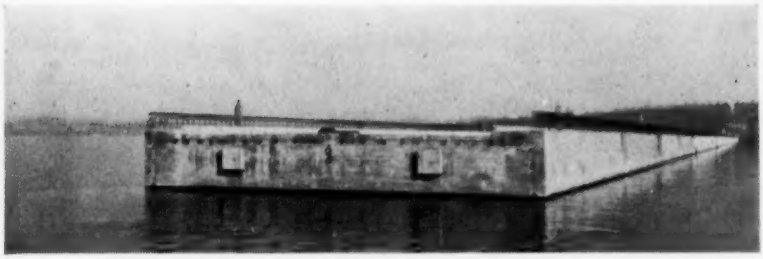

FIG. 8—SHEAR KEYS, BOLT HOLES AND SHEAR BAND IN END OF A PONTOON

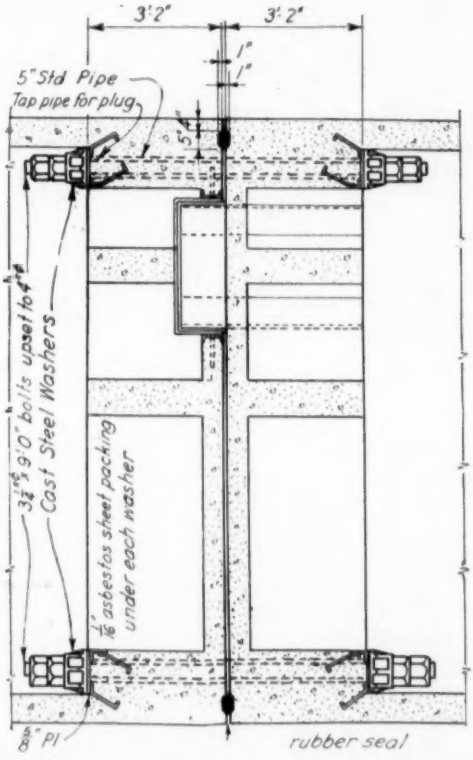

FIG. 9—VERTICAL SECTION THROUGH THE JOINT BETWEEN PONTOONS —SHEAR KEYS, BANDS AND RUBBER WATER STOP

system was devised. At one end of the pontoons two shear keys 3 x 3 ft. x 12 in. are centered 4 ft.-6 in. below the roadway surface on line with longitudinal webs. (May be seen in Fig. 8). At the opposite end of the pontoon are corresponding recesses. Both keys and recesses are metal faced. These form the means for initial connection. For

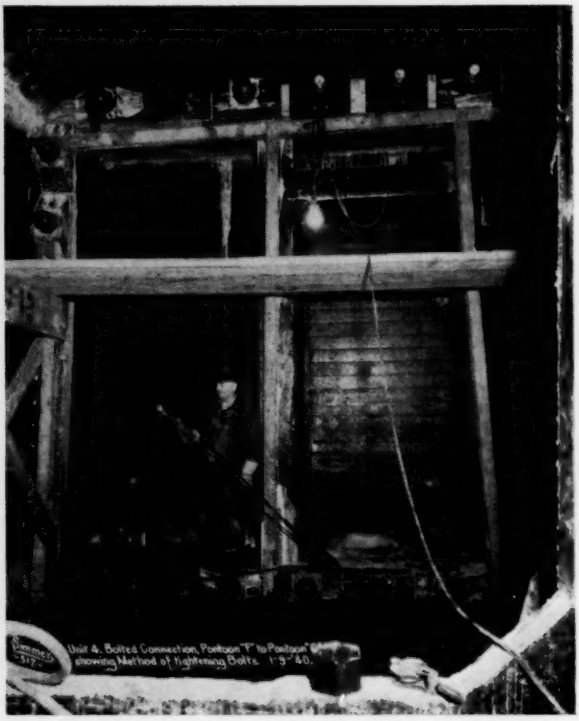

FIG. 10—THE BOLTED CONNECTION BETWEEN PONTOONS SHOWING METHOD OF TIGHTENING 3¼ IN. BOLTS

final connection 54 bolts are distributed around the outer margin of the sections and holes for these bolts, temporarily plugged, are provided through the ends of the pontoons. Also set in one end of the pontoon extending continuously across the bottom and up both sides is a ring of rubber which projects 2 in. (Fig. 9). When final connection is to be made and the adjoining sections have been brought together by their shear keys the upper plugs are knocked out and the upper row of bolts coming beneath the roadway slab are installed and tightened. (Fig. 10). By tightening these bolts the rubber ring is compressed and becomes a perfect water stop. The 1-in. space remaining between the pontoons is then unwatered, the remaining bolts are set and tightened and finally the joint is grouted. Extending around the bolt ring is a continuous recess 8 in. wide and 2 in. deep formed in both ends of the pontoons and when this is grouted it becomes a shear band extending continuously around the whole section. All told there are 54 bolts, 22 directly beneath the roadway slab, 22 directly above

the bottom slab and 5 on each side. These are all 3¼-in. bolts upset to 4 in. except 16 in the top row across the middle of the section which are 2¾ in. upset to 3½ in.

ANCHORAGE

The feature peculiar to a pontoon bridge is its anchorage system to resist the force of wind and wave and the first problem of design is to formulate a reasonable estimate of the magnitude of the wave to be designed for. There is considerable published data on deep sea waves; little or none on the waves in lakes. In the latter case the velocity of the wind to be anticipated, the configuration of the shore, the topography of the surrounding country, depth of water, "fetch" of open water, etc., make every lake a special problem. Our studies of Lake Washington led us to assume a trochoidal wave, 6 ft. high, 72 ft. from crest to crest and having a period of 2⅛ seconds running in any direction. This assumed wave is a highly conservative one for Lake Washington where even in strong winds the waves are short and choppy and of no great length. Because of the deep water at the bridge site no dynamic, horizontal action was assigned to the wave. The maximum effect of the assumed wave approaching the bridge broadside is equivalent to that of a uniform load of 2,000 lb. per lin. ft. of bridge acting for slightly over ⅓ second. It is apparent that rigorous and extended mathematical solutions are not warranted where fundamental assumptions are lacking in precision. Consequently the work of this periodical but momentary load was equated to that of a sustained continuous load. Combined with wind the horizontal load for which the anchor cable system is designed it consists of 520 lb. per lin. ft. uniformly along the bridge and, moving longitudinally along the bridge, 1,050 ft. on centers, a series of additional loads, each consisting of 380 lb. per lin. ft. uniformly for a length of 400 ft. and 400 lb. per lin. ft. uniformly for a length of 200 ft. The bridge itself acts as a horizontal stiffening truss to equalize and distribute the loads between the cables. Under this assumed loading 2¾ in. cold drawn wire anchor cables were required, these being spaced 350 ft. on centers and set on approximately a 3 to 1 slope (Fig. 11). These cables are attached to anchors set in the lake bed. Where the bottom is soft, as was generally the case, the anchor consisted of a large slab of reinforced concrete 14 ft. high and 26 ft. wide, shaped into two equal wings by a central fold or bend. From the middle of each wing on the concave side of the bend project deep sections of slab which reduce in vertical dimension toward the point of their convergence. At this junction point a steel eye bar is attached and at the far end of the eye bar the cable is fastened. This anchor is completely buried in the lake

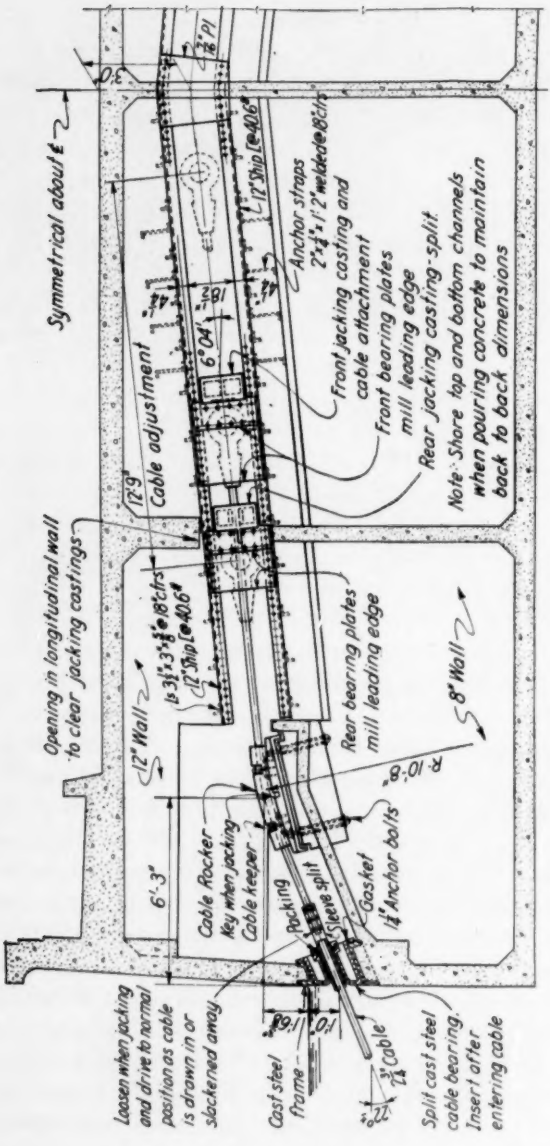
Symmetrical about ℄
Opening in longitudinal wall to clear jacking castings
12" Wall
6'-3"
Cable adjustment
6'-0"
12'-9"
Anchor straps 2"x½"x1'-2" welded @ 8 ctrs
Front jacking casting and cable attachment
Front bearing plates mill leading edge
Rear jacking casting-split
Note: Shore top and bottom channels when pouring concrete to maintain back to back dimensions
Cable Rocker
Key when jacking
Cable keeper
Packing
Sleeve split
Gasket
1¼" Anchor bolts
Rear bearing plates mill leading edge
8" Wall
R 10'-8"
Loosen when jacking and drive to normal position as cable is drawn in or slackened away
Cast steel frame
2½" cable
Split cast steel cable bearing. Insert after entering cable

FIG. 11—DETAIL OF TRANSVERSE CABLE ANCHORAGE

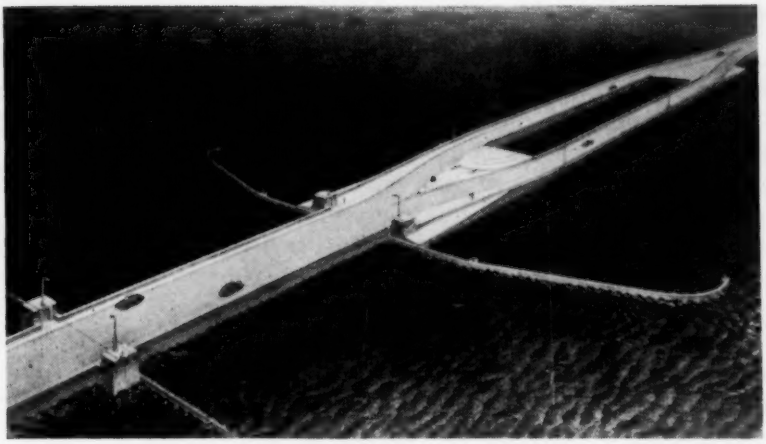

FIG. 12—ARTIST'S PERSPECTIVE OF PART OF LAKE WASHINGTON FLOATING BRIDGE, SHOWING THE DRAW STRUCTURE WITH RETRACTING SECTION

bottom, being jetted vertically downward into place. Where the bottom was hard the cables attach to large concrete boxes which were lowered into position and then were filled with and buried up by tremied gravel. The third type of anchor used in hard bottom consisted of double sets of steel piling driven into the lake bed. A pair of cables is attached at mid-length point of each 350-ft. pontoon. These enter the sides of the pontoons through stuffing boxes about 1 ft. below normal water surface. Inside the pontoon a concrete "drip pan" encloses the cable and extends 18 in. above the outside water surface so that any leakage which may occur at these points is limited. By a suitable jacking device provision is made within a pontoon for 12 ft.-9 in. of adjustment of cable length, thus providing for various tensions in the cable and for the 3-ft. annual change in water level in the lake.

RETRACTING SPAN FOR SHIP PASSAGE

Provision for the passage of large ships is made some 1500 ft. off of the easterly or Mercer Island end of the bridge. A 200-ft. opening of unlimited vertical height is formed by retracting a special pontoon section longitudinally into the waterway or slip formed by special flanking pontoon sections, the roadway offsetting and encircling this waterway. Fig. 12 shows the essential features of this arrangement. The movable section is drawn in and out by cables operated by machinery housed in the pontoons at the end of the waterway. In a minute and a half time the full 200 ft. of opening is made available.

The bridge was dedicated and put in service on July 2, 1940. Certain of its features are notable. Most striking is its great stability. Not the slightest movement can be felt even under strong winds. At all times it suggests the stability of solid ground. Its roadway is remarkably true, smooth and continuous. There are no offsets at the connecting joints and one is completely unaware when one passes from section to section. A third very interesting feature is the action of the waves which impinge against it. Instead of splash and spray against the sides of the pontoons the waves in all but the strongest winds are completely reflected and move backward away from the structure. At some distance out the reflected waves interfere with oncoming waves and it is in this zone that the water becomes exceedingly choppy and spray is thrown up into the air. With only the highest winds is spray blown over the railings.

ACKNOWLEDGMENTS

The Lake Washington Bridge was built by the Washington Toll Bridge Authority of which Lacey V. Murrow, Director of Highways, is member and chief engineer. R. B. McMinn, Admiral Luther E. Gregory and R. H. Thomson constituted the board of consulting engineers for the bridge project with the writer acting as chairman. Until after the award of contract Clark H. Eldridge was bridge engineer. R. M. Murray was bridge engineer during construction.

Construction proceeded under the direction of the writer. R. M. Murray was in direct charge of the entire Lake Washington Bridge project. G. A. Gregory was project engineer and L. R. Durkee was resident engineer inspector for the Public Works Administration.

The contract for the pontoon bridge construction was held jointly by the Puget Sound Bridge and Dredging Co., Seattle, Parker-Schram Co., Portland; J. H. Pomeroy & Co., San Francisco; and Clyde E. Wood of Los Angeles, operating under the name of Pontoon Bridge Builders.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June 1941.*

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of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JUNE, 1941*

Discussion of a paper by Charles E. Andrew:

Problems Presented by the Lake Washington Floating Bridge*

BY DUFF A. ABRAMS AND THE CONVENTION

BY MR. ABRAMS†

It may be of interest to place on record a few notes on a number of earlier engineering proposals somewhat similar to the Lake Washington floating bridge. Since 1800 there has been a series of agitations for a tunnel or bridge across the channel which separates southeastern England from the Continent. The narrowest width is about 20 miles, water depth up to 180 ft. Among the proposals for carrying a railroad across this portion of the English Channel were designs in which submerged tubes were used. This method was first proposed by a Frenchman, Mottray, in 1803. Thome de Gamond proposed a similar method, 1834.

James Wylson proposed to moor and buoy a tube at a uniform depth below the surface, 1855.

James Chalmers proposed a submerged tube on the bed of the Channel and published a book on the subject, 1861.

Bateman and Revy proposed another method for a submerged tube, 1869.

A method for constructing submerged tunnels patented in Italy was submitted to a Parliamentary Sub-Committee on the Channel Tunnel, 1929. This method consisted of articulated concrete sections encased in water-tight inner and outer shells of metal.

The above references are from "The Channel Tunnel Project" by R. A. Ryves, B. T. Batsford, London, 1929, and from the "Report of the Channel Tunnel Committee," London, 1930.

*JOURNAL Amer. Concrete Inst., Jan. 1941; *Proceedings* V. 37, p. 253.
†Consulting Engineer, New York.

None of the foregoing schemes were considered practical and did not carry much weight in competition with the proposals to build one or more tunnels in the chalk which underlies the English Channel at this point, however they are of considerable historic interest.

As early as 1873 an Anglo-French company was formed for the purpose of financing and constructing a sub-marine tunnel. In 1875 elaborate geological surveys and soundings were carried out and a route selected; shafts were sunk and pilot tunnels were run out under the water for over a mile from each shore The Channel tunnel project was again reviewed by English Parliament in 1930. To date all construction projects have been vetoed, ostensibly on the grounds of military defense.

The Lake Washington floating bridge may be visualized as a partly submerged tunnel, with the top levelled off for a trafficway. This design probably would not be satisfactory for the English Channel on account of the great length required, violent winds, waves, tides (up to 23 ft.) and currents, and the demands of navigation.

The engineers and contractors on the Lake Washington project are to be commended for the boldness of their design and the ingenuity with which the engineering problems were solved. They have taken advantage of the favorable conditions for a bridge of this type and have created a great public utility out of the blue prints of a bygone generation.

CONVENTION DISCUSSION

R. B. McMinn:* I quote the following from the public print to indicate some preconstruction reaction to the idea of a reinforced concrete pontoon bridge:

> Should the pontoon bridge actually be constructed, it is inconceivable that it would be usable in real storms. The structure must sink into every trough of the waves, and then, theoretically, it must rise with the following crests. Now a concrete float could not possibly act like a cork, so that the crests of the waves would be bound to smash right across it.
>
> Imagine such a structure staggering and lurching in the grip of a real storm. Imagine what would happen to automobiles trying to cross it.
>
> But, worst of all, imagine what would happen if the thing came apart.

In addition to the physical conditions favorable to the pontoon type of bridge which Mr. Andrew has enumerated the following are of interest: The total fluctuation of water level is 3 ft. and the level is definitely controlled by locks; the water in the lake is fresh water which has no detrimental effect on concrete as evidenced by examination of old piers and other concrete exposed to it; the pontoons are not exposed to ice as the lake has never frozen over its full width.

*Highway Bridge Engineer, Public Roads Administration. Mr. McMinn, who presented the paper to the convention, supplied additional information, chiefly in response to questions.

As pointed out by Mr. Andrew, there is but little water splashed on the roadway during storms, however, scuppers are provided at approximately 60-ft. intervals to drain the roadway of rain water and such water as may collect through splash or spray.

The question has been asked as to what means were provided for removing water from the cells in case some should get in. Leakage was not anticipated to be of any such extent as to require permanent pumping equipment for all of the pontoons. It was planned that, in the event of any leakage, the water would be removed by means of a portable pump. Mr. Andrew has stated, correctly, that leakage has not occurred and there are no indications that it will occur in the future. At this point I wish to emphasize that all cells in the pontoons are provided with access through manholes in the sidewalk.

Provision was made for 2-ft. expansion or contraction at the Seattle end of the bridge. The actual movement has not exceeded 5 in. The normal yearly range in temperature is from 30° in winter to 90°F. in summer. With half the height of the pontoon in water, the cooling effect of the water has considerable influence on temperature effects.

The arrangement providing for adjustment due to fluctuation of the water level consists of a combination of two methods; the primary method being a 150 ft. transition span at each end of the pontoon bridge. This provides very easy grades, however, the grade is further reduced by means of adjustment in the grade of approximately 900 ft. of the floating structure adjoining the transition span by pumping water in and out of the pontoons as necessary. The pumping device is automatically controlled, depending on the stage of water.

This pontoon bridge is the longest continuous bridge in the world; it is fully continuous, from lateral pressure for 6,561 ft., the full length of the floating structure; for vertical loads its greatest fully continuous length is about 4,000 ft. This feature is of decided advantage in increasing the stability of the bridge.

Mr. Andrew has not covered in his paper the subject of costs. This aspect is important for the primary reason that the relatively low cost of this pontoon bridge makes its construction possible from the economic standpoint; that is, if a conventional type bridge were considered (providing it were possible to construct one with such deep foundations as would be required) the cost would be so great that it could not be financed through revenue from tolls. The cost of the floating structure was approximately $500.00 per lin. ft. for the four-lane bridge with two sidewalks. This included the cost of the 200-ft. movable span.

In respect to financing, 30-year revenue bonds were issued supplementing the P. W. A. grant. The toll for a passenger car and driver is twenty-five cents, with five cents for each additional passenger. Receipts since the bridge was opened to traffic have exceeded the engineer's estimate and the income is in excess of that necessary to meet bond requirements. It is estimated that the bonds will be retired before the end of the 30-year period, when the bridge will be toll-free.

It is difficult to answer the question "What is the economic life of the pontoon bridge." In general, modern highway bridges are estimated to have a life of 50 years, but that does not mean many of them will not have a longer life than that. It is hard to conceive of this bridge not lasting more than 50 years. It is possible that the anchor cables might have to be replaced but that would be in the nature of a maintenance operation. It is like asking how long the Brooklyn Bridge will last; it was constructed in 1883 and is still in use. In any event 50 years is a long time and that is a conservative estimate.

This project presents an excellent opportunity for research, in respect to several features. The effect of wind and wave action on a bridge of this character is at present not susceptible to exact analysis. Measurements can be taken on the bridge to determine actual pressure from such causes under varying conditions; the effect of temperature can be determined and the deflection of the continuous pontoon bridge under various live loadings, with consequent effect on stresses in the concrete and steel, can be investigated. Such information will be of inestimable value when similar designs are undertaken.

The success of this bridge has been fully demonstrated and in my opinion it is destined to be world famous and the forerunner of other important pontoon bridges. It is the first concrete pontoon bridge to be constructed and it is the longest pontoon bridge in existence. It is the longest continuous bridge in the world. The movable span is both unique and economical.

The floating structure is very stable. The mass existent because of the use of concrete is a most important factor in aiding stability. Heavy trucks cause an almost imperceptible movement in the bridge and many people who ride over the bridge are not aware that it is a floating bridge. It has been subjected to a 65-mile wind with only the slightest movement resulting therefrom. In fact the structure, from all angles, has proven to be satisfactory and has fully met the expectation of the engineers responsible for it.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY, 1941*

A Twelve-Year Record of Concrete Mixtures*

By V. L. Glover†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Volumetric proportioning of aggregates for concrete was discontinued by the Illinois Division of Highways at the beginning of the 1930 construction season and the mortar-voids method of design adopted in its place; the aggregates were proportioned by weight and a system worked out of controlling the mixtures in the field. Accurate records of the proportioning and of the results obtained during the ensuing decade are compared here with records of volumetric proportioning of pavement concrete during 1928 and 1929.

After a period of development (1930 and 1931) of the mortar-voids method it was considered by 1932 entirely practical for use in large programs of construction. The marked improvement of the quality of the concrete during this period is reflected by an increase in flexural strength of about 35 per cent between 1929 and 1932, as shown by the average 14-day moduli of rupture of pavement concrete—far more than can be accounted for by any increase in the quality of the cements and is believed to have been accomplished without the use of additional cement. The high standards of 1932 have been maintained.

INTRODUCTION

From the standpoint of advancement in concrete technique, the year 1930 is undoubtedly the most important one in the annals of the Illinois Division of Highways. The old method of volumetric proportioning, with its attendant inaccuracies, was discontinued at the beginning of that year, except for a few hold-over jobs. In its place a method of designing mixtures was adopted which was based upon tests of the materials to be used, proportioning of the materials by weight, and controlling the uniformity of the concrete in the field. The method is still in use with numerous improvements in technique.

*Received by the Institute, Nov. 8, 1940.
†Engineer of Materials, Illinois Division of Highways.

It is not within the scope of this paper to describe in detail the method adopted. Briefly, however, it may be said that the mixtures are designed by the mortar-voids method developed by Professors Arthur N. Talbot and Frank E. Richart of the University of Illinois. The design methods are such that a concrete is produced which has a minimum modulus of rupture of not less than 650 p.s.i. at the age of 14 days. Rigid requirements control the accuracy of the devices for measuring water and for weighing aggregates. The amount of mixing water and the batch weights are adjusted as often as necessary to provide the correct amounts of water and saturated surface-dry aggregates in each batch, such adjustments being based on tests of the aggregates. As a general check, 6 x 6 x 30 in. beams are made from the concrete mixtures and tested at various ages to observe the gain in strength. For detailed information, reference may be made to Bulletin 137 of the Engineering Experiment Station of the University of Illinois, in which the basic principles underlying the method are given and to the Illinois Division of Highways "Manual of Instructions for Proportioning Engineers," in which the practical application of the method is worked out.

The engineers in charge of the proportioning and control of the concrete mixtures on the various jobs are required to keep complete records of the results of all tests performed, together with the adjustments in batch weights and the amounts of mixing water made on the basis of these tests, and to report these data, together with the results of concrete beam tests.

The data thus reported have been compiled and analyzed at the end of each construction season, and the results of these analyses, particularly those referring to the years immediately following 1930, have been invaluable in indicating those parts of the method which could be improved, thus aiding materially in developing the design methods to their present efficiency.

Summaries of data submitted from the various jobs have been kept from year to year and these reflect the improvement made in the quality of the concrete mixtures as a result of the method used. Fairly complete tabulations have been made for the work done directly by the State and some of these, being of general interest, are presented and discussed herein. Similar tabulations for work by the counties and cities under State supervision showing almost identical results are not included.

FLEXURAL STRENGTH OF THE CONCRETE

While the first field tests of concrete beams were made as early as 1923, field testing was not used before 1928 to the extent that it was

considered of sufficient interest to compile the results. From 1928, a fairly complete record of flexural strength of pavement concrete is available, and this provides a general index of its quality. Similar records are available for structure concrete beginning 1930. Table 1 is a summary of these records.

Considering the data representing the pavement concrete, it is seen from Table 1 that the 1930 results were somewhat discouraging. As a matter of fact the increase in strength between 1929 and 1930 was about the same as that between 1928 and 1929 as far as the 7, 14, and 21-day strengths are concerned; the 3 and 5-day strengths showed no improvement. During the first year the new method was in operation, therefore, it is questionable whether any definite improvement in quality was obtained, and any improvement which may have resulted was hardly commensurate with the added attention given to design, proportioning, and control of the mixtures.

These relatively unsatisfactory results however were not due to any fundamental unsoundness in the method of designing the mixtures. Proportions for one mixture only were given to the engineer in charge of each job, and while laboratory tests indicated that these proportions would produce a workable mixture having the required design strength when representative gravel and crushed stone coarse aggregates were used, too often the actual mixture was not workable. This left the field engineers with no other alternative than to modify the proportions by arbitrarily adjusting them until satisfactory workability was obtained. In many cases in making these adjustments the old volumetric proportions were used as a guide and the amount of mixing water was arbitrarily increased to obtain a consistency or workability which coincided with the engineer's individual opinion or idea as to what it should be. This procedure generally caused a reduction in strength. In these cases, therefore, the only improvement resulting from the use of the new method was the measurement of the aggregates by weight instead of by volume.

These deficiencies in the application of the new method were remedied successfully before the beginning of the 1931 construction. Instead of designing one mortar mixture of the required strength for each sand, a sufficient number were designed to cover the entire range in consistency which it was expected might be needed for the average job conditions. These design data were plotted in the form of charts to facilitate interpolation between the various designs and the charts were given to the engineers in charge of proportioning, thus enabling them to adjust the consistency of the mortar to obtain the desired workability.

While this modification more than any other helped to maintain the design strength, it was by no means the only modification necessary. The method of determining the amount of coarse aggregate which could be used with a given volume of mortar was found to be faulty and this was replaced by a more rational method. It was found also that gravel and crushed stone coarse aggregates required slightly different amounts of mixing water and adjustments were worked out accordingly. The methods adopted to adjust the amount of coarse aggregate and the quantity of mixing water are described later. Also, several minor modifications were made during 1930 and during the succeeding years based on information gained from experience.

The beneficial effects of these modifications to the method as first adopted are reflected in an increase in the strength of the concrete during 1931 and 1932 which are far greater than can be accounted for by any increase in quality of the cement during those two years. In fact, the average modulus of rupture during 1932, based on the 14-day tests, was nearly 35 per cent higher than the 1929 average.

The average 1932 strengths were maintained satisfactorily during several succeeding years with no further improvement of any consequence, indicating that practically the full benefit had been derived from the method employed. Slight increases in strength have been obtained during the last two or three years, which may be due to the activities of general field inspectors employed during this period.

Everything that has been said about pavement concrete is also essentially true of structure concrete for all years up to 1935, except that no comparison can be made between the present method of design and the method used prior to 1930. The concrete used in structures, however, showed somewhat lower strengths than that used in concrete pavements, due to the fact that many different consistencies are used in structure concrete, making control of the mixture more difficult, and it was entirely impractical to exercise full control of many small structures.

At the beginning of 1935, however, an improvement was noted which during the next few years brought the average strengths of concrete in structures much nearer to that of concrete in pavements. This improvement, coincident with the introduction of vibratory methods of placing concrete in structures, made wide variations in consistencies unnecessary and provided a greater measure of control.

STRENGTH COMPARISONS FOR GRAVEL AND CRUSHED STONE CONCRETE

When gravel and crushed stone mixtures contain the same amount of mixing water per bag of cement, it has been found that the crushed

TABLE 1—NUMBER OF BEAMS AND AVERAGE MODULUS OF RUPTURE IN POUNDS PER SQUARE INCH AT THE PRINCIPAL TEST AGES FOR THE YEARS 1928 TO 1939, INCLUSIVE

Year	3 Days		5 Days		7 Days		14 Days		21 Days	
	No.	M.R.	No.	M.R.	No.	M.R.	No.	M.R.	No.	M.R.
Pavement										
1928	6	400	35	545	103	570	52	600	16	647
1929	33	492	103	590	233	588	306	642	175	682
1930	991	453	1360	550	1876	609	4349	693	243	725
1931	751	546	925	638	1809	698	6019	786	489	840
1932	814	575	1469	690	2975	760	5277	864	244	927
1933	699	626	1313	715	1692	759	3426	855	2438	900
1934	437	605	977	725	1901	766	2017	854	858	902
1935	449	641	562	733	1857	772	1787	859	353	917
1936	325	616	517	723	1625	763	1468	863	470	955
1937	229	625	349	722	2053	780	1737	875	65	906
1938	365	689	470	780	1744	789	1416	882	95	902
1939	166	674	287	756	1194	813	1234	897	194	969
Structures										
1930	141	417	425	538	492	543	1101	653	67	710
1931	258	498	506	579	1349	665	3986	744	405	795
1932	614	521	1080	639	3033	716	5708	813	511	861
1933	248	573	538	686	1581	715	3771	810	1598	848
1934	73	558	295	687	1385	727	2860	818	217	829
1935	90	590	120	682	984	747	1604	847	104	887
1936	53	610	120	671	1803	747	2311	864	187	947
1937	40	649	71	684	1312	749	1611	864	83	897
1938	33	569	144	722	1137	761	1241	867	92	908
1939	71	579	95	724	1301	771	1208	869	59	875

stone mixtures generally produce the greatest strength. However, when the two are adjusted to produce approximately the same workability, which is necessary in field practice, nearly the same strength results. Adjustment for equal workability involves the use of a somewhat less absolute volume of coarse aggregate and a somewhat greater quantity of mixing water in the crushed stone mixtures, the quantities of sand and cement remaining the same. The method of adjusting the absolute volume of coarse aggregate will be described later.

Adjustment of the amount of mixing water is somewhat arbitrary, in that it is accomplished by visual inspection of the mixtures, supplemented by the slump test. It has been found that the amount of mixing water as determined in the laboratory is in nearly all instances the correct amount when gravel is used as the coarse aggregate. With crushed stone, it has been found necessary to allow mixing water in excess of that determined in designing the mixtures, varying from very small amounts up to about one-half gal. per bag of cement in a few instances, the exact amount depending upon the character of the crushed stone. Since it was considered of great importance to know the effect of using excess mixing water in the crushed stone mixtures, the flexural strengths submitted from the various jobs were compiled

separately for gravel and crushed stone. Table 2 is a summary of these data.

Considering first the data for the mixtures used in concrete pavements, it is seen that the average strengths of the crushed stone concrete are in most instances a little higher than the average for gravel concrete. This, however, is not true for all years and all test ages. During 1931, the situation was reversed; for the 21-day test age, the gravel concrete shows a higher strength about as often as the crushed stone concrete does, which suggests the possibility that the two types of concrete tend to become more nearly equal in strength as the age of the concrete increases. It appears, therefore, that the average strength of the two types of concrete is so nearly the same that gravel and crushed stone may be considered as having been used on an equal basis. This statement is in general confirmed by the results of tests made on cores taken from concrete pavements, usually at much later ages than the flexural tests.

The average strengths of the gravel and crushed stone concretes placed in structures are more nearly equal than the average of those in pavements. The difference for structure concrete is generally insignificant. The reason for the difference between pavement and structure concrete is not definitely understood, but it may be a natural consequence of the amount of attention now being given concrete mixtures. When concrete is placed continuously and under constant control as it is in pavement construction, the engineers in charge of proportioning seem to be reluctant to use excess mixing water in the crushed stone mixtures unless it is absolutely necessary and then only in very small amounts. Consequently, in order to obtain workable concrete the mixtures are adjusted, which usually results in the use of slightly more cement, accounting for the slightly greater strength of the crushed stone mixtures. This tendency has not been particularly noticeable where concrete is placed intermittently, often under minimum control as in construction other than pavement.

CEMENT FACTORS

While no accurate records were kept prior to 1931 as to the average amount of cement used per cubic yard of concrete, commonly expressed in barrels and termed the "cement factor," it is believed that the improvement in quality of concrete during 1930 and 1931 was obtained without increasing the average cement factor. Beginning with 1931, accurate records of cement factors are available—summarized in Table 3.

Table 3 shows that the improvement in quality of concrete during 1932 was accomplished without increasing the average cement factor

TABLE 2—NUMBER OF BEAMS AND MODULUS OF RUPTURE IN POUNDS PER SQUARE INCH OF GRAVEL AND CRUSHED STONE CONCRETE AT THE PRINCIPAL TEST AGES FOR THE YEARS 1931 TO 1939, INCLUSIVE

Year	Coarse Aggregate	3 Days		5 Days		7 Days		14 Days		21 Days	
		No.	M.R.	No.	M.R.	No.	M.R.	No.	M.R.	No.	M.R.
Pavement											
1931	Stone	299	551	303	627	798	698	2124	780	298	841
	Gravel	439	543	615	642	997	702	3810	789	173	859
1932	Stone	265	599	445	704	1263	788	2060	885	118	954
	Gravel	545	564	1023	685	1712	739	3197	850	126	904
1933	Stone	230	661	353	723	613	769	1218	880	905	928
	Gravel	469	609	958	714	1079	753	2206	841	1533	884
1934	Stone	162	616	426	755	810	791	813	875	290	898
	Gravel	275	599	551	701	1091	748	1204	841	568	905
1935	Stone	148	666	215	754	739	792	614	882	95	938
	Gravel	301	629	347	719	1118	758	1173	847	258	909
1936	Stone	58	664	165	742	422	782	391	884	141	969
	Gravel	267	606	352	714	1203	757	1077	856	329	948
1937	Stone	85	690	131	770	473	803	370	897	3	887
	Gravel	144	587	218	693	1580	773	1367	870	62	907
1938	Stone	205	727	244	837	801	806	529	891	15	925
	Gravel	160	642	226	719	943	777	887	876	80	898
1939	Stone	48	696	56	750	483	824	489	905	29	950
	Gravel	118	664	231	758	711	806	745	891	165	973
Structures											
1931	Stone	121	499	226	587	617	663	1514	740	175	800
	Gravel	137	497	280	573	732	667	2504	746	229	789
1932	Stone	156	506	267	640	1119	725	1927	815	228	876
	Gravel	458	526	813	638	1914	710	3781	812	283	849
1933	Stone	133	600	188	677	781	728	1441	825	548	881
	Gravel	115	541	350	691	800	703	2330	801	1050	831
1934	Stone	27	606	115	726	467	729	1148	833	42	888
	Gravel	46	530	180	662	918	726	1712	808	175	814
1935	Stone	45	583	48	658	508	743	723	858	33	941
	Gravel	45	596	72	698	476	751	881	838	71	862
1936	Stone	23	661	41	679	413	777	624	886	93	981
	Gravel	30	570	79	667	1603	736	1687	855	94	913
1937	Stone	15	670	18	679	533	734	702	853	43	894
	Gravel	25	636	53	686	779	759	909	872	40	899
1938	Stone	15	595	62	754	505	781	525	889	39	904
	Gravel	18	547	82	698	632	756	716	850	53	910
1939	Stone	6	546	10	729	500	771	500	867	14	911
	Gravel	65	582	85	723	801	771	708	871	45	864

over that used during 1931 and that the high standard of 1932 has been maintained by the use of about the same cement factor each year. In general it is seen that crushed stone concrete requires a slightly higher cement factor than gravel concrete and that structure concrete requires a somewhat higher cement factor than pavement concrete.

Table 3 also shows that the decreased cement factor of structure concrete incident with vibratory placing in 1934 and 1935 is small, suggesting that full advantage is not being taken of this aid to placement. It will be recalled, however, that the standard of structure concrete at that time was somewhat lower than that of pavement concrete and that improvement in methods of placing raised the standard of quality rather than reduced the cement factor and cost.

Since the mixtures are designed for a given strength and no definite cement factor is specified, the uniformity of the cement factors during the nine years for which records are available reflects the efficiency of the control exercised during the period.

TABLE 3—AVERAGE CEMENT FACTORS OR BARRELS OF CEMENT PER CUBIC YARD OF CONCRETE FOR THE YEARS 1931 TO 1939, INCLUSIVE

Year	Stone Coarse Aggregate		Gravel Coarse Aggregate		Stone and Gravel Coarse Aggregate	
	Cu. Yd.	Cement Factor	Cu. Yd.	Cement Factor	Cu. Yd.	Cement Factor
			Pavement			
1931	620,840	1.485	993,520	1.419	1,614,360	1.444
1932	511,720	1.485	959,640	1.419	1,471,360	1.442
1933	615,120	1.448	991,320	1.404	1,606,440	1.421
1934	355,960	1.438	443,300	1.398	799,260	1.416
1935	230,560	1.444	463,100	1.413	693,660	1.423
1936	146,520	1.444	361,680	1.411	508,200	1.421
1937	155,320	1.450	480,040	1.422	635,360	1.429
1938	242,880	1.436	281,600	1.419	524,480	1.427
1939	235,620	1.446	260,920	1.429	496,540	1.437
Total	3,114,540	1.459	5,235,120	1.414	8,349,660	1.431
			Structures			
1931	54,621	1.581	55,704	1.526	110,325	1.553
1932	108,974	1.531	129,288	1.567	238,262	1.551
1933	67,322	1.547	89,799	1.519	157,121	1.531
1934	69,983	1.524	113,003	1.484	182,986	1.499
1935	45,810	1.502	68,635	1.488	114,445	1.494
1936	40,176	1.528	110,422	1.488	150,598	1.499
1937	48,550	1.531	73,533	1.489	122,083	1.506
1938	41,509	1.506	52,777	1.478	94,286	1.490
1939	32,774	1.527	56,007	1.497	88,781	1.508
Total	509,719	1.532	749,168	1.508	1,258,887	1.518

Note: Cement factors are weighted averages.

MORTAR FACTORS

In a field investigation toward the end of the 1930 construction season to determine a satisfactory basis for selecting the amount of coarse aggregate which should be used with a given quantity of mortar, it was found that the amount depended chiefly upon the percentage of voids in the coarse aggregate. While other properties peculiar to aggregates from individual sources cannot be disregarded entirely, it is possible in any given instance to determine the quantity of coarse aggregate from the relation established in the field investigation mentioned and make minor adjustments by visual inspection, thus arriving at a mixture of satisfactory workability in minimum time. A chart showing the relation between the percentage of voids in the coarse aggregate and the amount of coarse aggregate which can be used per cubic foot of mortar, expressed in cubic feet absolute volume, has been used successfully since 1931.

The average relation between the percentage of voids in coarse aggregate and the absolute volume of coarse aggregate per cubic foot of mortar shown in this chart for a given class of work is not only a straight line, but is the particular straight line for which the quantity of mortar per cubic foot of coarse aggregate, dry rodded volume, is constant. This quantity of mortar has been termed the "mortar factor" and is inversely proportional to the amount of coarse aggregate used with a given volume of mortar. Since the mortar factor on the average is constant for any given class of work, regardless of whether the coarse aggregate is gravel or crushed stone, it has been found to be the most convenient medium for comparison of the amounts of coarse aggregate in the mixtures from year to year. Accurate records of mortar factors from the beginning of 1931 are summarized in Table 4.

Viewed from the standpoint of mortar factors it may be seen from Table 4 that essentially the same relation has existed between quantity

TABLE 4—AVERAGE MORTAR FACTORS OR QUANTITIES OF MORTAR PER CUBIC FOOT OF COARSE AGGREGATE, DRY RODDED VOLUME, FOR THE YEARS 1931 TO 1939, INCLUSIVE

Year	Stone Coarse Aggregate		Gravel Coarse Aggregate		Stone and Gravel Coarse Aggregate	
	Pavement	Structures	Pavement	Structures	Pavement	Structures
1931	.731	.778	.727	.762	.729	.769
1932	.767	.789	.739	.776	.749	.782
1933	.749	.790	.735	.782	.740	.785
1934	.742	.762	.738	.761	.740	.761
1935	.750	.760	.734	.768	.739	.765
1936	.734	.768	.728	.760	.729	.762
1937	.739	.767	.737	.779	.738	.774
1938	.739	.768	.735	.764	.737	.765
1939	.737	.762	.737	.769	.737	.766

of mortar and amount of coarse aggregate in the mixtures for gravel and crushed stone concrete each year during the entire period. The only exception of any consequence is found in the 1932 pavement mixtures where the mortar factor for crushed stone concrete is somewhat high. The reason for this is known definitely. The mortar factor for crushed stone concrete in 1931 was on the average somewhat low, resulting in occasional unsatisfactory results. The relatively high mortar factor the following year was the result of efforts on the part of the engineers in charge of proportioning to guard against similar unsatisfactory results.

Comparing the mortar factors from year to year, it is seen that, except for the 1932 crushed stone mixtures, approximately the same mortar factors have been maintained during the entire period as far as pavement concrete is concerned. The same is true of structure concrete beginning with 1934. The reduction in mortar factors for structure concrete shown for that year was due to vibratory methods of placing.

When it is considered that the determination of the amounts of the coarse aggregate to be used is not strictly a matter of design and that the exact amounts depend upon adjustments made by visual inspection, the uniformity of mortar factors which has been secured, especially during the last five years, is further evidence of the consistent proportioning.

CONCLUSIONS

From the data presented, the following conclusions may be drawn:

1—The adoption in 1930 of the mortar-voids method of designing concrete mixtures, the proportioning of materials by weight, and the control of the mixtures, together with the modifications introduced in 1931 and 1932 to adapt the method to large scale programs of construction, has resulted in considerable improvement in quality of concrete, as evidenced by the increase in flexural strength, which from 1929 to 1932 was about 35 per cent for the 14-day test age.

2—The increase in flexural strength is far greater than can be accounted for by any increase in the quality of the cements used during the period. It is believed that this increase was brought about without increasing the amount of cement used; this is known with certainty for that part of the increase between 1931 and 1932.

3—The strength of the concrete in structures was somewhat less than that of pavement concrete up to 1935, when vibration becoming more or less general, resulted in increasing the average strength of structure concrete until it was almost equal to that of pavement

concrete. In other respects, no marked increase in strength was observed after 1932, indicating that practically full benefit had been obtained from the new method at that time. The slight increase in strength during the last two or three years may have been due to the activities of general field inspectors employed during this period.

4—Although crushed stone concrete shows slightly higher strengths than gravel concrete in the majority of instances, the difference is so small that the two types of aggregate may be considered as having been used on an equal basis.

5—The average properties of the concrete mixtures evidenced by the compilations of cement and mortar factors, except for changes brought about by the introduction of vibration in placing structure concrete, show satisfactory uniformity from year to year and reflect the consistent proportioning.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the JOURNAL *for June 1941.*

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Discussion of a paper by V. L. Glover:

A Twelve-Year Record of Concrete Mixtures*

BY DUFF A. ABRAMS AND THE AUTHOR

BY MR. ABRAMS†

The principal conclusions of the paper with reference to the origin of the increased early strength of Illinois highway concrete during the 12 years ending with 1939, are not borne out by the data presented. Well-established facts concerning the increase in the quality of portland cements during this period readily account for the increased concrete strength, without any reference to the method of proportioning used.

Strength Gain Under Old Method of Proportioning

Mr. Glover calls attention to an increase in strength following the introduction of a new method of concrete proportioning in 1930; he failed to note the decided increase which occurred during the last year of the old method. From Table 1 we may compute the following increases in the beam strengths of pavement concrete during the last year the old method was used:

Age at Test days	Increase 1929 over 1928—%
3	23
5	8
7	3
14	7
21	6

The increase in strength in the last year under the old method compares favorably with the increase in concrete strength of typical Illinois cements tested at the same time in a Chicago laboratory (using an entirely different test procedure) and is not materially

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†Consulting Engineer, New York, N. Y.

different from the average annual increase during 4 or 5 years immediately following the introduction of the new method of proportioning by the Illinois Division of Highways. The increase can, in all cases, be traced to well-recognized changes in the fineness and composition of the cements.

Improvements occurred also in the methods of mixing, placing and finishing concrete in pavements which permitted the use of drier mixes. It is generally considered that a lowering of the water-cement ratio increases concrete strength; the paper gave no information on the effect of this factor, or whether any change in water-ratio of the pavement concrete occurred.

Widespread Increase in Concrete Strength

Students of concrete have noted the progressive increase in the early strength during the past 25 years. This increase has been general throughout the U. S. and other countries. It has recently become apparent probably for the reasons that test methods have become somewhat standardized and more tests have been reported than before. A closer examination of the subject shows that the improvement in cement quality has been continuous practically since the beginning of the manufacture of portland cement in this country. We have numerous concrete tests during the past 30 or 40 years; for earlier years we must depend largely on the physical tests and chemical analyses of the cements. The increase in early strength of cement which has been noted can be charged about equally to two factors:

(1) Increased fineness.

(2) Changes in compound composition.

Changes in Fineness of Cement

The facts with reference to changes in fineness of cement and its effect on the strength of concrete are too well known to require detailed treatment here. Early fineness tests were made with a No. 50 sieve; for a time the No. 100 sieve was used; for many years the 200 sieve was standard; later many laboratories used also a 325 sieve. During the past 5 years the 200 sieve has been dropped from cement specifications as too coarse for a satisfactory measure of fineness; most cements now show 90 to 98 per cent passing this sieve. Air separators or turbidimeters have almost entirely replaced sieves for fineness tests.

Probably the most exhaustive study of the effect of fineness of American cements on the strength of concrete was carried out by the writer at Lewis Institute (Proc. A.S.T.M., 1919). Clinkers from 7 different commercial mills were ground to 4 to 7 different finenesses ranging from 57 to 98 per cent passing the 200 sieve. For the usual range of mixes and consistencies the concrete strength increased for

each 1 per cent passing the 200 sieve (using 80 per cent passing as the basis for comparison) as follows:

Age at Test	Increase in Concrete Strength—%
7 days	2.5
28 "	2.0
3 mo.	1.7
1 yr.	1.4

Data recently collected by A.S.T.M. Committee on Cement shows the following increase in compressive strength of sand mortar for each 10 sq. cm. per gm. increase in specific surface of the cement; (Wagner turbidimeter values from 1760 to 1980):

Age at Test	Increase in Mortar Strength for Each 10 sq. cm.
3 days	1.9
7 "	1.5
28 "	1.2

The two groups of tests made over 20 years apart show the same tendency. The greatest increase came at early ages; the rate of increase dropped in proportion to the logarithm of the age.

In view of the general increase in fineness of cement throughout the U. S., I venture to guess there was a considerable increase in the fineness of Illinois cements during the 12 years covered by the paper.

Changes in Compound Composition

The compound in portland cement which is principally responsible for early strength is tricalcium silicate. The engineering profession became "compound conscious" about 1929, largely as a result of the researches of R. H. Bogue and his associates at the Bureau of Standards. Chemical analyses of the older cements show that the percentage of tricalcium silicate was steadily increasing long before the true significance of this change was realized. It came about in a purely rule-of-thumb fashion as a result of improved cement mill equipment, better kiln refractories which permitted burning at higher temperatures, to more careful preparation of raw materials, and to better control of mill operations.

Based on such records as are available on American portland cements, the writer plotted a diagram of 3CS against the year of cement manufacture. The points fall on a smooth curve; the following values were taken from this curve:

Year	3CS—Per Cent
1880	18
1900	28
1920	41
1930	48
1937	53

These values were based on ordinary standard portland cement; no high-early-strength cements were included. The average 3CS content increased about 0.5 per cent per year from 1877 to 1910; from 1910 to 1937 the rate was about 0.7 per cent per year.

It is difficult to determine the precise effect on concrete produced by changes in compound composition of the cement; such changes never occur singly, but are accompanied by changes in other compounds and generally by variations in fineness. Increase in 3CS is generally accompanied by a corresponding decrease in dicalcium silicate.

Cement manufacturers became interested in compound composition and for the first time became more tolerant of finer grinding of cement when it was pointed out that a high 3CS clinker ground much easier than one of low 3CS. It appears probable that the 3CS content of Illinois cements increased 8 to 10 per cent during the years covered by the paper.

A Chicago Laboratory Studied the Concrete-Making Qualities of Illinois Cements

While definite changes in fineness and composition cannot be attributed with certainty to a given cement at a given date, they are almost certain to apply to the average of a group of cements over a period of years. Fortunately it is not necessary to specu'ate on whether Illinois cements followed the usual trend during the first part of the period covered by the paper, since we have the results of an independent investigation *which was made at the time with the same cements.* In 1916 an investigation was begun in Chicago under the writer's direction for the sole purpose of studying changes in the concrete-making qualities of the same cements from time to time.

For the major part of our work at the Structural Materials Research Laboratory, Lewis Institute, a blend of 4 or 5 brands of the same cements was used. The cements were purchased from local dealers. Each lot of cement as well as the blend was subjected to physical and chemical tests; one group consisted of 6 x 12-in. concrete cylinders made from each blend for test at ages of 7 days to 10 years. Sand and gravel from the same source and of a fixed grading up to 1½ in. was used throughout, the same quantity of cement (1-5 mix by volume), also the same water-ratio (.54 by weight) and the same methods of mixing, molding and curing. During the first 15 years the specimens were made by the same men. Controllable factors were the same from lot to lot and year to year, except the cement.

A summary of the first 15 years of these laboratory tests was published in the *Engineering News-Record*, Apr. 20, 1933. That report

shows an age-strength curve for 12 lots of the blend tested during the 5 years 1926-30 (mid-year 1928); these values can be compared with the 1928 tests by Mr. Glover. Values are given also for 3 lots for 1930-31; these can be compared with the average for these two years by Mr. Glover. This comparison gives the following increase in concrete strength for 2½ years 1928 to 1930-31, using here only the values from 7 to 28 days:

Tests by	Specimen	Increase in Strength—Per Cent					Average Age days
		7d.	14d.	21d.	28d.	Av.	
Chicago Laboratory	Cyl.	30	—	—	23	26½	17½
Illinois Highways	Beam	—	23	23	—	23	17½

The above years and ages were selected purposely to cover the *period of transition* from the old to the new method of proportioning by the Illinois Division of Highways and to include the 14-day test age which is emphasized in the paper.

The strength increases at average age of 17½ days are about the same for the two groups of tests; the somewhat higher precentage for the Chicago tests is due to the use of the 7-day value which, as shown below, gave a higher-than-average increase.

Fig. A shows a more complete comparison between these two groups of tests from 3 days to 6 months. A single straight line to a log-scale of ages represents both groups. Except for erratic values at 5 and 7 days, all tests might have been parts of the same series—a remarkable concordance when we consider the wide differences in aggregates, method of loading and the conditions under which the concrete was manufactured and the tests made. Both groups of tests show that the greatest increase in concrete strength came at the earlier ages.

Increase in Strength of Illinois Highway Concrete

The score stands about as follows, based on 14-day tests in Table 1:

Method of Proportioning	Period	Average Annual Increase in 14-day Strength—Per Cent
Old Method	1928-1929 (last year)	7
Transition	2½ yr. Chicago tests, (Fig. A)	10
"	2½ yr. Highway " "	9½
New Method	1 year 1929 to 1930	8
" "	2 " " " 1931	11
" "	3 " " " 1932	11½
" "	4 " " " 1933	8
" "	5 " " " 1934	6½
" "	6 " " " 1935	6
" "	7 " " " 1936	5
" "	8 " " " 1937	4½
" "	9 " " " 1938	4
" "	10 " " " 1939	4

Increase in Concrete Strength of Illinois Cements
During 2½ Years, 1928 to 1930-31
Increase in Strength in 2½ Years
50
40
30
20
10
0
o Illinois Highways, Beam Tests
x Chicago Laboratory, Cylinder Tests
3 5 7 14 21 28 days 3mo. 6mo. 1yr. 2yrs. 5yrs.
Age at Test — Log. Scale

FIG. A—INCREASE IN CONCRETE STRENGTH OF ILLINOIS CEMENTS DURING TWO AND ONE-HALF YEARS—1928 TO 1930-31

At 3 different places in the paper attention is directed to the increase of 35 per cent in the modulus or rupture during the 3 years 1929 to 1932. It will be noted that this gives the highest average annual increase recorded; it probably represents also the years of greatest increase in 3CS in cement and the most pronounced increase in fineness. During 1929-32 the majority of cement manufacturers were giving unusual attention to attempts to improve their product in order to meet the new competition from the high-early-strength cements. The paper does not state whether any high-early-strength cements were included.

Let us assume for the sake of argument that one-half of the increase in concrete strength was due to increased fineness of the cement. We saw above that 14-day mortar strength increased 1.35 per cent for each 10 sq. cm. increase in fineness. The annual increase in specific surface necessary to account for one-half the 35 per cent increase in concrete strength in 3 years will be:

$$\frac{.35 \times 10}{3 \times 2 \times .0135} = 43 \text{ sq. cm. per gm.}$$

It seems to me that this is not an unlikely figure for average annual increase in fineness during this period. The value of the paper would be enhanced if Mr. Glover gave average cement fineness for each year, weighted in accordance with the number of tests reported for a given cement.

Conclusions

The above values show that the annual increase in Illinois highway concrete strength was practically uniform:

(1) Under the old method of proportioning.
(2) During the transition from the old to the new method.
(3) During first 4 or 5 years of the new method.

The strength increase in Illinois highway concrete, tested in beams, was parallel with that of compression tests of cylinders carried out at Chicago at the same time on the same cements.

It is apparent that the increased strength is confined principally to the early history of the concrete and becomes insignificant after about 1 year.

The paper gave no information on the composition or fineness of cements used; however it appears that Illinois concrete merely kept pace with the improved quality of the cements and owed nothing to the new method of proportioning; all the evidence indicates that the concrete strength would have shown the same year-to-year increase if the old method had been continued.

Judged by the results given in the paper, and our knowledge of changes which have occurred in the properties of cement, we can find no justification for the added cost of the new method of proportioning.

* * *

My remarks should not be construed as disparaging the high quality of Illinois highway concrete. Concrete giving moduli of rupture of 800 to 900 p.s.i. at 14 days, even when tested in short beams, is undoubtedly of satisfactory quality. We should not be confused as to the factors which have contributed to the marked increase in the early strengths during the past dozen years.

AUTHOR'S CLOSURE

Mr. Abrams' discussion is directed almost entirely toward refuting the statement made in the paper that "the increase in flexural strength is far greater than can be accounted for by any increase in the quality of the cements used during the period," meaning the period 1929 to 1932, inclusive. His conclusions are summarized in the statement that "all the evidence indicates that the concrete strength would have shown the same year-to-year increase if the old method had been continued," which implies indirectly that be believes that the State Illinois is spending money unjustifiably to control the quality of concrete.

The statement to which Mr. Abrams takes exception was based on the results of the tests made on the cements actually shipped and used in construction jobs in Illinois during the period mentioned. The number of tests made on these cements ranged from 13,000 to 15,000 per year during the four-year period from 1929 to 1932, inclusive, and were not less than 7,700 during any one year of the twelve-year period from 1928 to 1939, inclusive. The number of tests, therefore, is sufficient to carry considerable weight in any discussion and the results should indicate with certainty the trend of any change in quality of cement. Table 5 included as a part of this discussion shows the average 7-day and 28-day standard briquet strengths for each year of the twelve-year period, the percentage of increase in the average

TABLE 5—AVERAGE STANDARD BRIQUET STRENGTHS AND COMPARISON OF YEAR-TO-YEAR INCREASE IN STRENGTH OF BRIQUETS AND PAVEMENT CONCRETE

Year	No. of Cars Tested	Amount of Cement (Barrels)	Briquet Strength (Pounds)		Per Cent Increase in Strength			
					Briquets		Concrete	
			7-Day	28-Day	7-Day	28-Day	7-Day	14-Day
1928	20,502	5,027,687	307	402	.....	.....	....	....
1929	12,984	3,250,348	314	411	2.3	2.2	3.2	7.0
1930	13,504	3,434,441	325	417	3.5	1.5	3.6	7.9
1931	14,990	3,830,971	338	427	4.0	2.4	14.6	13.4
1932	15,351	4,005,478	340	435	0.6	1.9	8.9	9.9
1933	12,540	3,254,089	338	431	−0.6	−0.9	−0.1	−1.0
1934	9,514	2,367,187	339	425	0.3	−1.4	0.9	−0.1
1935	7,703	1,947,927	349	433	2.9	1.9	0.8	0.6
1936	8,554	2,086,846	349	436	0.0	0.7	−1.2	0.5
1937	8,398	2,042,960	354	443	1.4	1.6	2.2	1.4
1938	8,023	1,961,414	363	450	2.5	1.6	1.2	0.8
1939	8,718	2,064,788	367	446	1.1	−0.9	3.0	1.7
Total	140,781	35,274,136	4,083	5,156	18.0	10.6	37.1	42.1
Av.	11,732	2,939,511	340	430	1.64	0.96	3.37	3.83
1929 to 1932, inclusive					8.3	5.8	29.3	34.6
1932 to 1939, inclusive					7.9	2.5	7.0	3.8

The number of plants shipping cement for highway construction during any one year was not less than 10 nor more than 19. The number of tests of the cement may be considered equal to the number of cars shipped, as seldom more than one test was made for each car.

The percent increase in strength of concrete is calculated from the data for pavement concrete in Table 1.

briquet strength from year to year, and the percentage of increase in the average pavement concrete beam strength from year to year, calculated from the 7-day and 14-day data given in Table 1 of the paper.

The standard briquet test has been specified by the American Society for Testing Materials as an acceptance test for cements for a long period of years and should provide a true indication of the quality of cements. Assuming that this is the case, Table 5 shows that during the twelve-year period there was an average improvement in the quality of the cements, measured by average year-to-year increases in briquet strength, of 1.64 per cent and 0.96 per cent for the 7-day and 28-day tests, respectively. During this same period there was an average year-to-year increase in the strength of the concrete of 3.37 per cent and 3.83 per cent for the 7-day and 14-day tests, respectively, or more than twice as much for the concrete as for the briquets, based on the 7-day tests. Considering individual years, it is seen that the peak of the improvement in cements did occur between the years 1928 and 1932, coincidental with the period of development of the Illinois method of proportioning as stated by Mr. Abrams. But the increases in concrete strength for the years 1931 and 1932 were 14.6 per cent and 8.9 per cent, respectively, based on the 7-day tests, as compared with corresponding increases in briquet strength of 4.0 per cent and 0.6 per cent. Considering the increases in the briquet strength and concrete strength during the period 1929 to 1932, inclusive, on the

basis of the direct comparison provided by the 7-day tests, it is seen that the briquet strength increased 8.3 per cent, while the concrete strength increased 29.3 per cent, an increase of more than 3.5 times as much. It is not illogical to conclude, therefore, that improvement in the quality of cements used was responsible for less than one-third of the increase in the 7-day concrete strength obtained during this period.

There are no undue differences between the values representing the improvement in cements and those representing the improvement in concrete for years subsequent to 1932. Based on the 7-day tests, the briquet strength increased 7.9 per cent and the concrete strength 7.0 per cent during the entire period 1932 to 1939, inclusive. Although the 28-day briquet strength and the 14-day concrete strength data cannot be compared directly, they do show small but approximately similar percentages of increase during that period. These data show that subsequent to the period of development of the Illinois method of concrete design and proportioning, improvement in quality of cements and increase in the strength of field concrete kept pace with each other almost exactly. The fact that the 7-day briquet and concrete strength data show higher percentages of increase than do the 14-day and 28-day data undoubtedly indicates that most of the improvement after 1932 may be attributed to improvement in the high early strength properties of the cements.

It seems certain, therefore, that the improvement in the quality of the cements during the period of development of the Illinois method of proportioning was far less than the improvement in the quality of the concrete during the same period, and that improvement in quality of the cements and the concrete thereafter kept pace with each other within reasonable and explainable variations. This is still more apparent from the graphical illustrations in Fig. B and C.

Fig. B shows the percentages of year-to-year increase in strength from Table 5 for the 7-day test age. It is seen that these increases are very nearly the same for the briquets and the concrete for all years except 1931 and 1932. For these two years the percentage of increase for the concrete far exceeds the percentage of increase for the briquets, showing that the improvement in cements was far from sufficient to account for the improvement in the concrete, whereas for all other years the improvement in quality of cements and concrete kept fairly well in pace with each other.

Fig. C shows the briquet strengths from Table 5 and the pavement concrete beam strengths from Table 1 for the 7-day test age. It is seen that a considerable increase in concrete strength occurred during

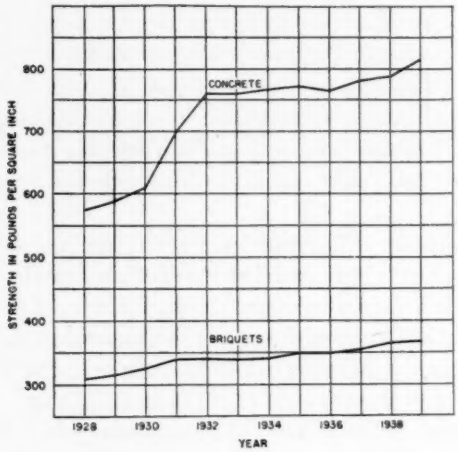

FIG. B—COMPARISON OF IMPROVEMENT IN QUALITY OF CEMENTS AND CONCRETE AS MEASURED BY THE PERCENTAGE YEAR-TO-YEAR INCREASE IN BRIQUET AND BEAM STRENGTHS FOR THE 7-DAY TEST AGE

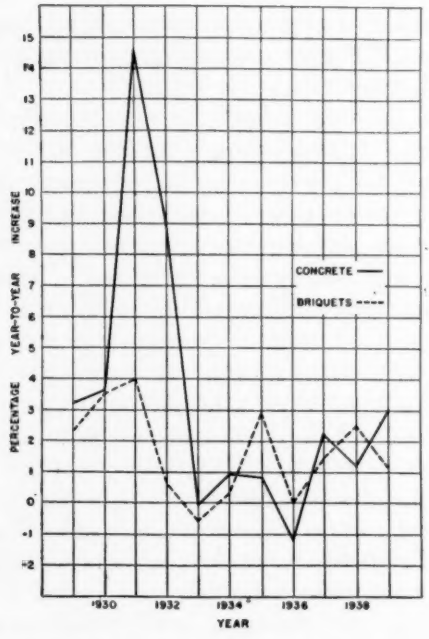

FIG. C—COMPARISON OF THE AVERAGE YEARLY BRIQUET AND BEAM STRENGTHS

the years 1931 and 1932 not paralleled by the briquet strength. Except for this, the two curves parallel each other fairly well. This shows that the strength of the concrete increased at a far greater rate in 1931 and 1932 than did the quality of the cements. Comparison for other test ages would show similar results, the 7-day test age having been selected only because it provides a direct comparison.

It should be apparent therefore that the statement to which Mr. Abrams takes exception was not mere expression of opinion or belief. It is not intended to imply that similar results could not have been obtained by some other method of design and proportioning receiving the same degree of attention and strict enforcement of the specifications, but they would not have been obtained had the methods in use before 1930 been continued.

The strength data upon which Mr. Abrams' discussion is based are not given in sufficient detail to arrive satisfactorily at the reason why he should obtain results so far different from those shown in Table 5. The miniature charts in the Engineering-News Record, April 20, 1932, referred to by Mr. Abrams are unsatisfactory for checking the percentages shown by him. It is to be regretted that he did not present also the entire data for the years subsequent to 1931. Inasmuch as the four or five brands of cement represented by Mr. Abrams' data were purchased in the Chicago area, these same brands were undoubtedly used in highway construction in Illinois, but during the twelve-year period covered by the data presented in the paper, not less than ten brands of cement were used during any one year. It is entirely possible that the four or five brands mentioned by Mr. Abrams were not truly representative of all of the cement actually used in Illinois, whereas the data in Table 5 represent all of the cement actually shipped by the various plants, weighted exactly in accordance with the number of cars of each brand shipped. Mr. Abrams does not give information as to the number of test specimens represented by the data which he presents, but it is safe to assume that the number was exceedingly small compared with the total of more than 140,000 tests represented by the data in Table 5, an average of nearly 12,000 tests per year, made under actual working conditions.

The decided increase in concrete strengths between the years 1928 and 1929 was mentioned in the paper and it was observed that this increase was about the same as between the years 1929 and 1930. In 1928 the testing of concrete beams was not done generally on all jobs, as may be seen from the small number of specimens represented for that year in Table 1. It is not known how nearly the 1928 tests

presented all of the pavement concrete placed during that year. Some of the increase shown, but apparently not all of it, may be accounted for by improvement in cements, and the control of production of concrete in 1929 may also have been better than in 1928.

Nothing was said in the paper as to whether the water-cement ratio was lowered when the new method of proportioning was introduced, because nothing was definitely known in regard to this, It may be assumed with some certainty, however, that prior to 1930 the mixtures were seldom if ever on the dry side, because the checking of water measuring devices was not required and there was no control over the amount of mixing water used other than adjustments made by visual inspection of the concrete mixtures, supplemented by an occasional slump test. It is quite certain that the degree of control exercised during 1931 and later resulted in a reduction in the water-cement ratio and that this reduction was responsible for some of the gain in concrete strengths. Except in unusual cases, the water-cement ratio now ranges from 5.1 to 5.6 gal. per bag of cement, depending upon the characteristics of the aggregates used.

Mixtures in which high-early-strength cement was used and those in which high-early-strength was obtained by using increased amounts of standard portland cement were not included in the data presented. The use of the 14-day test data for computation of increase in strength had no connection whatever with the fact that this test age showed the highest increase in strength. The specifications under which all of the concrete was produced required that the mixtures be designed for a specified 14-day strength and the various tables show that the greatest number of tests were made at this age. It became customary, therefore, to use the 14-day test age for comparison wherever possible, not only because it was of the most interest from the standpoint of the design of the concrete mixtures, but also because the 14-day strengths should provide the most accurate comparison.

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Weathering Resistance of Concretes Containing Fly-Ash Cements*

By
RAYMOND E. DAVIS, HARMER E. DAVIS AND J. W. KELLY†
MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This investigation, extending the scope of an investigation previously reported, was undertaken to determine whether fly ashes of moderately high carbon content could be used advantageously as puzzolanic replacements for small percentages of portland cement, particularly with regard to weathering resistance of concrete. Eleven fly ashes varying in carbon content (1 to 17 per cent) and fineness (specific surface 2500 to 5500 square centimeters per gram) were each admixed with a modified portland cement, with 10 and 20 per cent of cement replaced by an equal weight of fly ash. The fly-ash cements thus produced were tested to determine the water requirement, time of setting, autoclave expansion, tensile strength of standard mortar, compressive strength of concrete, drying shrinkage of concrete, and resistance of saturated concrete to the action of repeated freezing and thawing. Herein the performance of the fly-ash cements is compared with that of the portland cement, and the influence of carbon content, fineness, and amount of fly ash is shown. On the whole, the use of fly ash resulted in concretes of quality equal or superior to that obtained with the portland cement alone.

INTRODUCTION

In an investigation previously reported to the Institute,[1] it was found that fly ashes of low carbon content and high fineness are excellent puzzolanic materials when used as replacements, by weight, for as much as 30 per cent of portland cement and—under mass-concrete conditions—for as much as 50 per cent of the portland

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†Respectively Professor, Associate Professor and Lecturer in Civil Engineering, University of California, Berkeley.
[1] "Properties of Cements and Concretes Containing Fly Ash," by Raymond E. Davis, Roy W. Carlson, J. W. Kelly, and Harmer E. Davis, JOURNAL, Amer. Concrete Inst., May-June 1937 (*Proceedings*, Vol. 33), pp. 577-612.

TABLE 1—SOURCES OF FLY ASH

Fly Ash	% Carbon	Specific Surface, sq. cm/g.	Source	Plant
Commonwealth Edison	1	5530	Commonwealth Edison Co., Chicago, Ill.	Fisk Station
Chicago	2	4150	Chicago District Electric Generating Corp., Chicago, Ill.	State Line Station
Duquesne	7	2700	Duquesne Light Co., Pittsburgh, Pa.	Colfax Power Station
Rochester A[a]	8	3850	Rochester Gas and Electric Corp., Rochester, N. Y.	Station 3
Dayton	9	2980	The Dayton Power and Light Co., Dayton, Ohio	Miller's Ford Station
Detroit	12	3220	The Detroit Edison Co., Detroit, Mich.	Trenton Channel Plant
Philadelphia	12	2920	Philadelphia Electric Co., Philadelphia, Pa.	Schuylkill Station
Newark	13	4980	Public Service Electric and Gas Co., Newark, N. J.	Essex Generating Station
Rochester B[b]	14	4240	Rochester Gas and Electric Corp., Rochester, N. Y.	Station 3
Consolidated Edison	16	2470	Consolidated Edison Co. of New York, New York, N. Y.	....
Indiana	17	3920	Northern Indiana Public Service Co., Michigan City, Ind.	Michigan City Generating Station

[a]Boilers operating at normal load.
[b]Boilers operating at low load.

cement. The investigation, although extensive, was not sufficiently inclusive to answer the following questions:

1. What is the effect of carbon content alone? (Generally the high-carbon fly ashes of the earlier investigation were of low fineness.)

2. Does high carbon content of fly ash lead to low weathering resistance of concrete?

3. Can fly ashes of relatively high carbon content be employed in small percentages of replacement?

The present investigation was designed to answer these three questions, particularly with regard to weathering resistance. In the previous investigation, the test for weathering resistance consisted of only thirty cycles of relatively slow freezing and thawing, followed by a compression test to determine a "strength ratio" of treated to untreated specimens. In the present investigation, the test for weathering resistance involved short cycles of rapid freezing and thawing of saturated concrete; the cycles were continued until the specimens (3 by 6-in. cylinders) lost 25 per cent in weight through disintegration. This latter test, more severe and more revealing than that used in the former investigation, is generally regarded as a

more reliable index to weathering than was the type of test formerly used.

Materials and Test Methods

From ten different sources were obtained eleven fly ashes having a wide range of carbon content and fineness (Table 1). There were four groups of fly ashes; in each group the carbon content was approximately constant (1-2, 7-9, 12-14, and 16-17 per cent carbon). Within each group there were two or more fly ashes differing greatly in fineness. The samples did not necessarily represent the regular product of the plants.

For each fly ash, two fly-ash cements were tested: One contained 10 per cent, by weight, of fly ash and 90 per cent of portland cement; the other contained 20 per cent of fly ash and 80 per cent of portland cement. The fly ash and portland cement were blended by being admixed in a steel drum equipped with steel blades.

The portland cement with which the fly ashes were blended for test was of the "modified" or moderate-heat-of-hardening type (56 per cent tricalcium silicate, 16 per cent dicalcium silicate, 6 per cent tricalcium aluminate, and 16 per cent tetracalcium alumino-ferrite). It was ground to a specific surface of 2360 sq. cm. per gram as computed on the basis used in this investigation, or 2180 sq. cm. per gram on the A. S. T. M. basis.

Information regarding the test conditions is given in the footnotes of the tables of test results.

TEST RESULTS

Chemical Analysis

Table 2 shows a wide range in chemical composition of the fly ashes. The chemical analysis was also computed on a loss-free carbon-free basis, in order to permit study of the mineral portion of the fly ash; although space does not permit its inclusion here, such an analysis can readily be computed from the values of Table 2.

Fineness

The particle-size distribution and specific surface of the fly ashes was determined by the hydrometer method, with water (and a dispersing agent) as the suspending medium. The sample weight was corrected for the solubility of a portion of the fly ash in water (see Table 2, column 15), and the determinations of density and viscosity of the suspending medium included the effect of the soluble portion of the fly ash.

Fly ashes have a relatively high percentage of particles in the smaller and therefore most important sizes with regard to surface area (Table

TABLE 2—CHEMICAL ANALYSIS OF FLY ASHES

Fly Ash	Chemical Analysis, Per Cent													
	Carbon[b]	SiO_2	Al_2O_3[c]	FeO[d]	Fe_2O_3	CaO	MgO	SO_3	Ign. Loss (1800°F.)	$Na_2O + K_2O$	Insol. Res.	Free Lime	Water-Soluble Alkalies as Sulfates	Total Water-Soluble
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Commonwealth Edison	0.99	44.96	19.36	3.03	16.45	6.06	1.26	3.38	3.60	1.02	68.85	0.18	0.31	5.79
Chicago	1.56	45.24	20.71	3.20	16.04	6.29	1.09	2.85	2.46	2.66	75.84	0.27	0.97	6.76
Duquesne	7.00	45.46	28.10	2.61	11.22	2.19	0.91	0.70	7.26	0.66	87.52	0.58	0.29	1.67
Rochester A	7.84	43.32	27.64	2.90	11.73	4.08	1.09	1.07	8.43	0.45	80.59	0.55	0.19	2.04
Dayton	8.75	51.38	31.36	1.68	4.29	1.58	0.40	0.53	8.76	0.90	91.42	0.00	0.34	1.19
Detroit	11.53	45.46	28.87	1.51	6.55	2.04	0.28	0.89	11.78	0.96	88.83	0.00	0.27	1.90
Philadelphia	11.55	37.76	29.45	3.70	13.56	2.50	0.48	0.73	11.66	0.18	81.51	0.00	0.25	1.43
Newark	12.51	35.60	28.30	2.86	12.86	4.84	0.87	1.01	12.74	0.58	69.82	0.23	0.25	2.26
Rochester B	13.64	39.50	24.26	2.10	13.18	3.44	0.57	2.17	15.00	0.33	84.88	0.00	0.31	4.08
Consolidated Edison	16.38	36.90	26.10	3.58	12.39	2.00	0.80	0.92	16.55	0.24	83.14	0.00	0.28	1.06
Indiana	17.39	34.56	25.33	2.82	9.40	5.82	0.94	1.61	17.86	1.92	78.49	1.04	0.45	1.57
Avg. for 11 fly ashes	9.92	41.83	26.32	2.73	11.61	3.71	0.79	1.44	10.55	0.90	80.99	0.26	0.36	2.70
None (portland cement)[a]	....	20.20	5.51	....	5.33	64.47	1.75	1.57	1.01	....	....	0.80	....	....

[a]Compound composition: 56% 3CS, 16% 2CS, 6% 3CA, 16% 4CAF.
[b]Determined as loss on ignition at 1500°F., based on oven-dry weight (oven-drying at 230°F.)
[c]Includes all of ammonia group except iron.
[d]Total Fe and ferrous compounds as FeO.
NOTE: Fly ashes dried at 230°F. before analysis.

3, columns 6 and 7). In order to obtain as nearly as possible a true picture of the relative finenesses, it was necessary to compute the specific surface on a basis which involved several increments within the range below 10 microns and which took account of the material finer than 1 micron. The specific surface of the fly ashes on this basis ranged from 2500 to 5500, and averaged 3700 square centimeters per gram (Table 3, column 8). The portland cement had a specific surface of 2360 sq. cm. per gram on this basis as compared with 2180 sq. cm. per gram on the A. S. T. M. basis.

Specific Gravity

The specific gravity of the eleven fly ashes ranged from 2.08 to 2.61 and averaged 2.33 (Table 3, column 3). There was no consistent relation between specific gravity and carbon content of fly ash, or between specific gravity of fly ash and the performance of the fly-ash cements in mortar and concrete.

Water Requirement

For the various fly-ash cements, the water requirement to produce normal consistency of neat-cement paste is shown in Table 4, column 6; and the water requirement to produce a fixed consistency of concrete is shown in Table 5, column 6. For fly ashes of low carbon content (1 and 2 per cent), the water requirement (of concrete) was generally lower than that for the portland cement, and was about the same for

TABLE 3—FINENESS AND SPECIFIC GRAVITY OF FLY ASHES

Fly Ash	% Carbon	Specific Gravity	% Passing Sieve (Dry)		% Finer Than[b]		Specific Surface, Sq. Cm. / Gram[b,c]
			No. 200 (74 Microns)	No. 325 (44 Microns)	10 Microns	1 Micron	
1	2	3	4	5	6	7	8
Commonwealth Edison	1	2.61	98.6	97.5	55.2	3.9	5530
Chicago	2	2.54	98.8	98.0	54.1	0.9	4150
Duquesne	7	2.22	78.0	71.3	26.0	0.8	2700
Rochester A	8	2.33	95.6	91.4	40.0	2.2	3850
Dayton	9	2.08	89.4	85.9	31.2	0.0	2980
Detroit	12	2.16	89.7	86.1	38.0	0.0	3220
Philadelphia	12	2.38	89.2	85.4	34.5	0.0	2920
Newark	13	2.42	95.4	91.5	43.5	3.9	4980
Rochester B	14	2.30	96.2	92.8	52.0	0.0	4240
Consolidated Edison	16	2.25	90.3	84.6	28.5	0.0	2470
Indiana	17	2.30	92.9	88.7	39.8	0.9	3920
Avg. for 11 fly ashes	10	2.33	92.2	88.5	40.3	1.1	3720
None (portland cement)[a]	..	3.18	97.2	90.7	36.8	1.5	2360[d]

[a]Commercial cement reground in laboratory.
[b]By hydrometer. For fly ashes, sedimentation in water; for portland cement, sedimentation in kerosene.
[c]Computed from particle-size distribution, using increments between 80, 60, 50, 40, 30, 20, 10, 5, 2, 1, and 0 microns.
[d]On A.S.T.M. basis, 2180 sq. cm. per gram.
NOTE: Fly ashes dried at 230°F. before test.

TABLE 4—RESULTS OF STANDARD TESTS ON PASTES AND MORTARS

Fly Ash			Fly-Ash Cement		Neat Paste[b]				Standard Mortar		
						Time of Setting, Hr.:Min. (Gillmore)			Tensile Strength, p.s.i. (Briquets)		
Name	% Carbon	Spec. Surf., sq. cm/g.	% Fly Ash	Spec. Surf., sq. cm/g.	Normal Cons., % Water by Wt.	Initial	Final	Autoclave Expansion, %	7 Da.	28 Da.	6 Mo.
1	2	3	4	5	6	7	8	9	10	11	12
....	..	....	0	2360[a]	23.5	1:30	2:35	0.062	350	420	390
Commonwealth Edison	1	5530	10	2680	23.4	2:00	3:05	....	345	405	460
			20	2990	23.3	2:05	3:20	0.034	345	435	485
Chicago	2	4150	10	2540	22.7	1:40	2:45	.....	370	430	460
			20	2720	22.8	2:10	3:30	0.030	350	415	495
Duquesne	7	2700	10	2390	24.5	1:40	2:35	.....	315	395	460
			20	2430	26.5	1:50	2:50	0.031	295	395	470
Rochester A	8	3850	10	2510	25.0	1:35	2:45	.....	350	435	475
			20	2660	27.0	2:05	3:20	0.022	315	410	505
Dayton	9	2980	10	2420	25.0	2:00	3:10	.....	365	415	460
			20	2480	27.0	2:05	3:05	0.011	305	400	505
Detroit	12	3220	10	2450	25.5	1:45	2:45	.....	350	430	495
			20	2530	27.5	2:05	3:05	0.018	315	425	505
Philadelphia	12	2920	10	2420	24.6	1:40	2:45	.....	365	415	465
			20	2470	26.0	2:05	3:05	0.015	310	385	440
Newark	13	4980	10	2620	25.0	1:55	2:45	.....	340	410	490
			20	2880	26.5	2:25	3:25	0.025	310	395	480
Rochester B	14	4240	10	2550	26.0	1:55	3:05	.....	340	435	490
			20	2740	28.5	2:20	3:45	0.020	335	415	515
Consolidated Edison	16	2470	10	2370	26.2	1:50	2:45	.....	305	375	420
			20	2380	28.5	2:35	3:50	0.026	300	390	465
Indiana	17	3920	10	2520	24.5	1:45	2:50	.....	360	415	455
			20	2670	25.7	2:05	3:15	0.037	305	395	495
Avg. for 11 fly ashes	10	3720	10	2500	24.8	1:50	2:50	.....	345	415	465
			20	2630	26.3	2:10	3:20	0.024	315	405	485

[a]On A.S.T.M. basis, 2180 sq. cm. per gram.
[b]All cements sound in standard steam test on neat-cement pat.

a 20-per cent replacement as for a 10-per cent replacement. For fly ashes of medium and high carbon content, the water requirement (of concrete) was quite generally higher than that for the portland cement, and was higher for a 20-per cent replacement than for a 10-per cent replacement. Beyond this, there was no consistent relation between carbon content and water requirement. In general, the finer the fly ash the lower the water requirement.

Time of Setting

Although the fly-ash cements set more slowly than the portland cement with which they were admixed, the times of setting were within the limits permitted by standard specifications (Table 4, columns 7 and 8). The greater the percentage of fly ash the slower the setting. The time of setting was not affected greatly or consistently by carbon content or fineness of fly ash.

TABLE 5—STRENGTH, DURABILITY, AND VOLUME CHANGES OF CONCRETES

Fly Ash			Fly-Ash Cement		W/C, by Wt.	Compressive Strength, p.s.i.[b,d]			No. of Freezing Cycles Required[b,e]		Length Change, Millionths[e,f]			
									For Increase in Length of 1%	For Loss in Weight of 25%	Expansion in Fog	Net Contraction (Storage in Air After 28 Days)		
Name	% Carbon	Spec. Surf., sq. cm/g.	% Fly Ash	Spec. Surf., sq. cm/g.		7 Da.	28 Da.	1 Yr.			28 Da.	35 Da.	56 Da.	6 Mo.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
.....	...	....	0	2360[a]	0.42	4250	5360	6050	92	136	30	275	395	515
Commonwealth Edison	1	5530	10	2680	0.40	4480	5740	6930	179	208	15	265	360	460
			20	2990	0.40	4190	5390	7080	284	317	20	260	355	450
Chicago	2	4150	10	2540	0.42	4180	5220	6730	123	158	20	280	400	495
			20	2720	0.40	4090	5380	6970	208	237	25	245	380	470
Duquesne	7	2700	10	2390	0.44	4320	5470	6630	103	146	25	275	440	520
			20	2430	0.46	3420	4800	6190	107	146	35	275	440	525
Rochester A	8	3850	10	2510	0.44	4410	5470	6490	88	130	25	275	440	525
			20	2660	0.45	3820	5200	6790	195	212	15	290	415	470
Dayton	9	2980	10	2420	0.44	4250	5600	6970	202	226	10	310	450	530
			20	2480	0.45	3710	5130	6650	229	240	20	295	420	475
Detroit	12	3220	10	2450	0.44	4180	5280	6770	176	188	20	305	455	540
			20	2530	0.46	3710	5040	6410	212	226	25	325	470	550
Philadelphia	12	2920	10	2420	0.43	4450	5590	6720	201	210	25	295	430	535
			20	2470	0.44	3950	5030	6320	187	198	25	335	460	545
Newark	13	4980	10	2620	0.42	4610	5820	7020	237	266	30	285	395	490
			20	2880	0.43	3760	5240	6220	344	360	35	325	435	540
Rochester B	14	4240	10	2550	0.44	4430	5670	6790	171	188	25	290	460	560
			20	2740	0.45	3890	5490	6490	167	187	30	285	435	520
Consolidated Edison	16	2470	10	2370	0.44	4270	5420	6560	193	217	35	300	430	535
			20	2380	0.46	3500	4890	5970	187	200	30	340	495	585
Indiana	17	3920	10	2520	0.42	4460	5670	6680	246	259	45	280	415	525
			20	2670	0.43	3920	5410	6820	279	307	40	295	425	515
Avg. for 11 fly ashes	10	3720	10	2500	0.43	4370	5540	6750	174	200	25	285	425	520
			20	2630	0.44	3810	5180	6540	218	239	25	295	430	515

[a]On A.S.T.M. basis, 2180 sq. cm. per gram. [b]3 by 6-in. cylinders. [c]1½ by 1½ by 12-in. bars. [d]Standard curing.
[e]Specimens standard-cured for 5 mo., then subjected (while continuously saturated) to cycles of freezing at −15°F. and thawing at 70°F. First 144 cycles were two 4-hr. cycles (2 hr. freezing, 2 hr. thawing) per day; remaining cycles were four 2-hr. cycles (1 hr. freezing, 1 hr. thawing) per day.
[f]Specimens standard-cured in fog to 28 da., then stored at 70°F. in air of 50 per cent R. H.; all length changes referred to length at 1 dav.
Note: Cement-aggregate ratio 1:5.6 by wt.; 0 to ¾-in. gravel; flow 60 per cent.

TABLE 6—RATIO OF STRENGTH, FLY-ASH CEMENT TO PORTLAND CEMENT

Property		Ratio, Fly-Ash Cement to Portland Cement							
		10% Fly Ash				20% Fly Ash			
		7 Da.	28 Da.	6 Mo.	1 Yr.	7 Da.	28 Da.	6 Mo.	1 Yr.
Tensile strength of mortar	Max.	1.06	1.04	1.27	....	1.00	1.04	1.32	....
	Min.	0.87	0.89	1.08	....	0.84	0.92	1.13	....
	Avg.	0.99	0.99	1.19	....	0.90	0.96	1.24	....
Compressive strength of concrete	Max.	1.08	1.09	....	1.16	0.99	1.02	....	1.17
	Min.	0.98	0.97	....	1.07	0.80	0.90	....	0.99
	Avg.	1.03	1.03	....	1.12	0.90	0.97	....	1.08

Soundness; Autoclave Expansion

All of the cements were sound, as indicated by the standard steam test on a neat-cement pat.

In the recently adopted standard autoclave test for soundness of cement, the expansion of the fly-ash cements was only $\frac{1}{5}$ to $\frac{2}{3}$ that of the portland cement (Table 4, column 9). The greatest expansion of any fly-ash cement was 0.04 per cent.

The greater the lime content or the free-lime content of fly ash, the greater the autoclave expansion. There was no marked effect of FeO or carbon. To a slight extent, the finer the fly ash the greater the expansion.

Strength

For the various fly-ash cements, Table 4 (columns 10 to 12) shows the tensile strength of standard mortar at ages up to 6 months, and Table 5 (columns 7 to 9) shows the compressive strength of standard-cured concrete at ages up to 1 year. As between fly-ash cements containing a given percentage of fly ash, the range in strength was generally small. The ratios given in Table 6 afford comparisons between the fly-ash cements and the portland cement, with regard to both compressive and tensile strengths; and Fig. 1 shows the compressive strengths graphically.

At ages of 7 and 28 days, the fly-ash cements containing 10 per cent of fly ash developed concrete strengths which were about the same as, or somewhat higher than, those for the portland cement (Fig. 1). At the age of 1 year, all of these fly-ash cements were considerably stronger than the portland cement. The 20-per cent fly-ash cements were relatively low in concrete strength at the early age of 7 days, but at the age of 28 days they were approaching or exceeding the portland cement and at the age of 1 year ten of the eleven cements exhibited higher concrete strengths than did the portland cement.

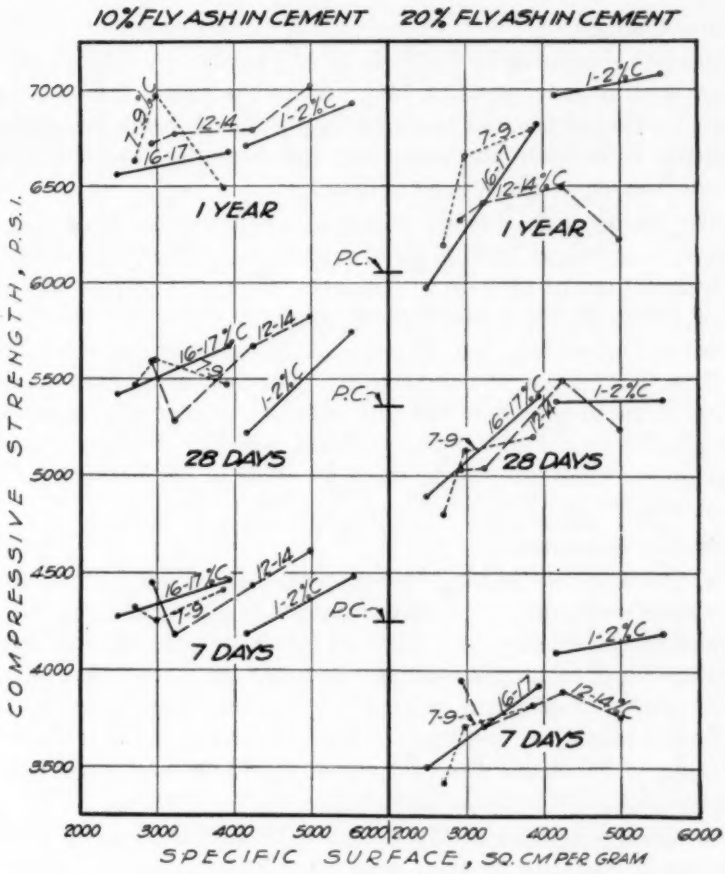

FIG. 1—EFFECT OF FINENESS, CARBON CONTENT, AND PERCENTAGE OF FLY ASH ON COMPRESSIVE STRENGTH OF STANDARD-CURED CONCRETE

The strength of concrete was generally lower for the fly-ash cements containing 20 per cent of fly ash than for the 10-per cent replacements. There was no consistent relation between carbon content of fly ash and strength of concrete. With a few exceptions, the finer the fly-ash the stronger the concrete. Of the fly-ash cements containing a given percentage of fly ash, those of relatively low water requirement generally exhibited relatively high strengths of concrete.

In general, the higher the strength the greater the resistance to the action of freezing and thawing. However, the range in strength is so small that it would be impossible to predict durability from strength with any reasonable degree of precision.

Volume Changes

Concrete specimens in the form of 1½ by 1½ by 12-in. bars were stored continuously moist at 70°F., up to the age of 6 months. The expansion ranged from 30 to 65 millionths, there being no significant difference as between the various fly-ash cements. For the portland cement, the expansion was 50 millionths.

Other concrete bars were standard-cured for 28 days and then exposed to drying at 70°F. in air of relative humidity 50 per cent. The net contractions at ages up to 6 months, referred to the length at the age of 1 day, are shown in Table 5, columns 13 to 15. For fly ashes of carbon content less than 10 per cent, the contraction was generally less than that for the portland cement. For fly ashes of carbon content greater than 10 per cent, the contraction was generally about 5 per cent greater than that for the portland cement. On the whole, the effect on contraction of using these fly ashes in percentages up to 20 was small.

Weathering Resistance

The freezing-and-thawing test employed in this investigation was one which is generally considered to provide a satisfactory indication of resistance to weathering. The test involves cycles of rapid freezing and thawing of saturated concrete, and it is far more severe than the usual natural weathering where the freezing and thawing occur slowly and much less frequently. For this reason, the test was begun at the age of 5 months, when the concrete had developed a reasonably large proportion of its ultimate strength.

The concrete specimens (3 by 6-in. cylinders) were cured moist at 70°F. for 5 months and then were alternately frozen at -15°F. and thawed at 70°F., while continuously saturated, until the specimens had lost 25 per cent through disintegration. At intervals during the period of treatment, the specimens were inspected, weighed, and measured to determine the linear expansion.

The number of freezing-and-thawing cycles required to produce a 25-per cent loss in weight is shown in Table 5, column 11. The other measures of disintegration—expansion (column 10) and appearance rating—correlated fairly well with the loss in weight, although there was a tendency for the concretes of relatively low weathering resistance to disintegrate by softening throughout whereas those of relatively high resistance finally failed by disintegration from the exterior.

Fig. 2 shows the effect of fineness, carbon content, and percentage of fly ash on the number of cycles of freezing and thawing required to produce a 25-per cent loss in weight of concrete; it also affords com-

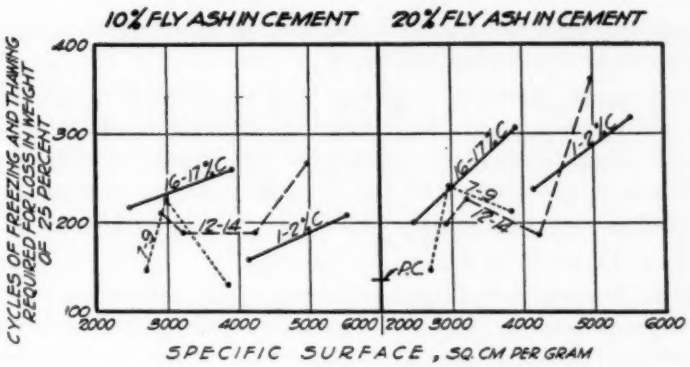

FIG. 2—EFFECT OF FINENESS, CARBON CONTENT, AND PERCENTAGE OF FLY ASH ON DURABILITY OF CONCRETE

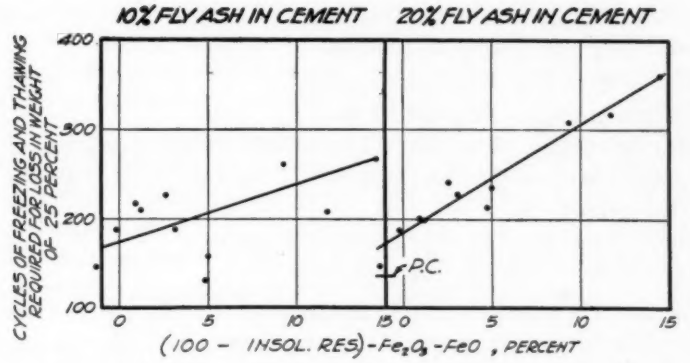

FIG. 3—EFFECT OF ACID SOLUBILITY OF FLY ASH (EXCLUSIVE OF IRON COMPOUNDS) ON DURABILITY OF CONCRETE

parison of the fly-ash cements with the portland cement. Only one fly-ash cement exhibited a lower resistance than the portland cement and the difference (4 per cent) was slight. On the other hand, one fly-ash cement required 2.65 times as many cycles as the portland cement. On the average, the fly-ash cements containing 10 per cent of fly ash were 1.47 times as resistant as the portland cement while the fly-ash cements containing 20 per cent of fly ash were 1.76 times as resistant as the portland cement.

In general, the finer the fly ash the greater the resistance to the action of freezing and thawing (Fig. 2). As indicated by the relative positions of the carbon-group lines in the diagrams, there was no consistent effect of carbon content; this lack of correlation indicates

that high carbon content in itself (within the limits of the tests) is not detrimental to weathering resistance.

Of the many chemical properties and combinations of properties which were investigated to determine if possible a relation between chemical composition and durability, the closest correlation was obtained with the portion of the fly ash, exclusive of the oxides of iron, that was soluble in dilute (1:3) hydrochloric acid, or

$$(100\text{—Insol. res.})\text{—}Fe_2O_3\text{—}FeO$$

That is, generally, the greater this solubility index the higher the weathering resistance (Fig. 3). The index also correlated fairly well with strength and drying shrinkage of concrete—the greater the solubility the higher the compressive strength and the lower the shrinkage.

CONCLUSIONS

The following general conclusions apply to the range of conditions of these tests, which were as follows:

Eleven fly ashes, from ten sources.
Carbon content of fly ash: 1 to 17 per cent.
Specific surface of fly ash: 2500 to 5500 sq. cm. per gram.
Replacement of portland cement by fly ash: 10 and 20 per cent, by weight.
A single modified type of portland cement.
Ages of test: various, up to 1 year.

It became apparent from the test results that no single characteristic, or simple combination of characteristics, controls the performance of a fly ash in concrete.

1. As compared with the portland cement, the fly-ash cements exhibited considerably lower autoclave expansions; about the same drying shrinkage of concrete; and—with minor exceptions—considerably greater resistance of concrete to the action of freezing and thawing.

2. Nine of the eleven 10-per cent fly-ash cements were somewhat stronger in concrete than the portland cement at the early ages of 7 and 28 days; and at the age of 1 year all of these fly-ash cements were considerably stronger than the portland cement. The 20-per cent fly-ash cements were relatively low in concrete strength at 7 days, but they approached or equaled the portland cement at 28 days and (with one exception) exceeded it considerably at the age of 1 year.

3. The percentage of replacement of portland cement with fly ash had little effect on shrinkage. The fly-ash cements containing 20 per cent of fly ash were generally somewhat lower in strength but were

generally more resistant to weathering than the 10-per cent replacements.

4. The carbon content of fly ash showed no consistent effect on soundness, strength, shrinkage, or weathering resistance. Thus it appears that, within the limits of the tests, carbon content in itself was not detrimental to concrete quality.

5. As compared with fly ashes of relatively low fineness, those of high fineness generally exhibited a lower water requirement to produce a fixed consistency of paste or concrete, slightly greater autoclave expansion, greater strength of concrete, about the same drying shrinkage, and higher resistance of concrete to the action of freezing and thawing.

Suitability of Fly Ash

From the results of these investigations it appears that fly ashes of fairly high carbon content but of reasonably high fineness may be used in small percentages as a replacement for portland cement, with the probability that—with proper conditions of curing and normal conditions of exposure—such use will lead to an improvement in the quality of concrete, particularly as regards weathering resistance, over that which could be obtained with the portland cement alone. Although these investigations were limited to one portland cement (of the modified type), study of the results of the previous investigation indicates that similar improvement may be expected with the other common types of portland cement—high-early strength, normal, and low heat.

Within the limits of fineness and carbon content of fly ash covered by this investigation, and for percentages of replacement up to 20, it is believed that the suitability of a given fly ash can be ascertained by the standard physical tests for cement, that is, by determining the time of setting, soundness (by autoclave), and tensile or compressive strength of mortar for a cement containing the fly ash under consideration.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by April 1, for publication in the* JOURNAL *for June 1941.

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JUNE, 1941*

Discussion of a paper by Davis, Davis and Kelly:

Weathering Resistance of Concretes Containing Fly-Ash Cements*

BY J. S. NELLES†

We are indebted to the authors of this paper for the thoroughness of their investigation. The fact that the carbon content of the fly-ash does not have a bad effect on the weathering resistance of concrete, when the fly-ash is used to replace 10 and 20 per cent of the portland cement, is very reassuring. It is to be hoped, however, that they will continue their good work and investigate concrete mixtures containing greater amounts of fly-ash than are included in this report; not considering the fly-ash entirely as a replacement for portland cement.

We have been investigating the use of fly-ash in concrete mixtures for the last 12 years and have actually placed more than 12,000 cu. yds. of fly-ash concrete in structures of various types, some of which have been in service, exposed to the weather, for many years. The fly-ash content ran as high as 800 lb. per cu. yd. of concrete.

Professor Davis' investigations have covered the use of fly-ash as a replacement for portland cement, and other investigators have reported on its use as an admixture. In our investigations we have gone further and added fly-ash in greater amounts.

An example of replacing cement with an excessive amount of fly-ash is given in Table I. The five-bag mix was at that time the standard mix for concreting around underground electric cable conduits. The other mix was experimental, and represented one transit-mix truck-load of three cubic yards.

*JOURNAL, Amer. Concrete Inst., Jan. 1941; *Proceedings* V. 37, p. 281.
†Engineer, The Detroit Edison Co., Detroit.

TABLE 1

Cement Content Bags/Cu. Yd.	Water-Cement Ratio Gals/Bag	Replacing Cement With An Excessive Amount of Fly-Ash (Lb.)	Compressive Strength at 28 Days Aver. of 5 Spec. (p.s.i.)
5	7.12	0	4030
4	8.97	150	4470

*Made as comparative batches.

TABLE 2

Cement Content Bags per Cu. Yd.	Water-Cement Ratio Gals. per Bag	Fly-Ash Replacing Fine Aggregate Only (Lb.)	Compressive Strength (p.s.i.)	
			28 Days	1 Year
4*	8.4	0	2060	
4*	10.7	400	2550	
4	16.0	790	1884	3980

Examples using fly-ash as fine aggregate are given in Table 2.

A comparative example, using fly-ash as replacement for part of cement and fine aggregate, is given in Table 3.

TABLE 3

Cement Content Bags/Cu. Yd.	Water-Cement Ratio Gals/Bag	FLY-ASH			Compressive Strength (p.s.i.)		
		Replacing Cement (Lbs.)	Replacing Fine Aggregate (Lbs.)	Total Lb.	28 Days	3 Mo.	6 Mo.
7	4.8	0	0	0	4765	5715	6585
6	6.0	100	200	300	5500	7055	7435

In all of our work the carbon content of the ash has been 7½ per cent or greater, which corroborates the findings of the paper under discussion, even though we have incorporated fly-ash into the mix in quantities greatly in excess of those reported.

There seems to be an ever-increasing demand for more fines in the fine aggregate, and these extra fines cannot be found in the natural or manufactured sands. We believe fly-ash to be an ideal material for this purpose, and that its biggest field in concrete mixes will be as a replacement for a considerable part of the fine aggregate.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY, 1941*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information. No formalities; just write a letter to "the Editor."

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

How to design concrete mixtures? (35-21)*

The accompanying tabulation (page 298) may be of interest to the engineer who is weighing the pro's and con's of different types of fine aggregate for concrete. The data are auxiliary to, and tied in with, the data in Table 4, page 587, *Proceedings*, Vol. 35. The ratio of fine to total aggregate is 38 per cent for concrete mixes. The coarse aggregate is 2 in. crushed limestone having a fineness modulus of 7.86, and the fine aggregate is as indicated.—JOHN C. SPRAGUE†

†Development Engineer, Dravo Corporation, Pittsburgh.
*See June 1939 JOURNAL, p. 585.

SOME COMPARISONS BETWEEN CRUSHED STONE SAND AND NATURAL SAND

	Natural Sand and Gravel	Blend of 80% Natural Sand and 20% L. S. Sand	Natural Sand		Sand-stone Sand	Limestone Sand	
Fineness Modulus (fine aggregate)	2.97	3.25	3.25	2.97	3.24	2.82	3.24
Fineness Modulus (Combined aggregate)	6.3±	6.1±	6±	5.8±	6±	5.8±	6±
% Passing No. 100 Sieve	2.0%	3.0%	1.3%	2.0%	10.0%	7.0%	10.0%
Per cent flow of mortar for water-cement ratio of 0.68	18.3%	20.3%	....	18.3%	....	4.6%	....
Water-cement ratio for a mortar flow of: 20%	0.72	0.73	0.70	0.72	....	0.83	....
100%	0.87	0.87	0.83	0.87	1.07	1.09	1.09
Cement factor for water-cement ratio of 0.90, slump of 2½ in.	1.1	1.3	1.3	1.3	1.6	1.5	1.6
Total mixing water per cubic yard of concrete for a slump of 2½ in. and remolding effort of 20 drops	30 gal.	34 gal.	35 gal.	35 gal.	42 gal.	41 gal.	42½ gal.
Water-cement ratio required in concrete for a slump of 2½ in. and where the cement factor is 1.3	0.77	0.87	0.90	0.90	1.09	1.07	1.09
Compressive strength of concrete for a C. F. of 5.3, slump of 3½ in. and water-cement ratio of:	....	0.87	....	0.90	....	....	1.09
7-day	....	2811 p.s.i.	....	2862 p.s.i.	....	....	2442 p.s.i.
28-day	....	3976 p.s.i.	....	4023 p.s.i.	....	....	3206 p.s.i.
90-day	....	....	....	4408 p.s.i.	....	....	3681 p.s.i.

NOTE: All mortar is made of sand passing a No. 4 sieve and the maximum size of all coarse aggregate in the concrete is 2 in., the fineness modulus being 7.86. Limestone is used as coarse aggregate, except as indicated in the first column.

Why does concrete sometimes stick to metal forms? (37-66)

Q—Several instances have come to our attention in which concrete has stuck to metal forms and for which there appeared to be no explanation readily available. These instances have been in products plants (particularly vault manufacturers) as well as building work, after entirely successful use of the forms and of the same oil for some time.

We tried a number of different types of form oil of varying consistency, but with little effect. Yet in each case for apparently no explainable reason the sticking of the concrete to the forms stopped.

If you have information that might help explain the condition, it will be appreciated.

A—The experience recited in the question is not unusual. Probably every type of form oil for metal forms, even when apparently used appropriately, has failed at times without evident reason. Sometimes the explanation will be found in insufficient cleaning of the forms. Abrasive means of cleaning that reveal bright metal will promote

sticking although the forms are perfectly clean and properly oiled. Concrete sliding or falling against the forms may be sufficient to remove the oil and permit sticking.

Although equally inexplicable stopping of the sticking trouble may occur, the sticking is by no means certain to stop, or to stop promptly, and positive action to insure its stopping is usually desirable whether the cause is positively known or not. After it is certain that there are no causes for the sticking which should otherwise be corrected, the trouble may be eliminated in most cases where it is practicable to do so, by first properly cleaning and oiling the forms with non-drying oil and then exposing the oiled surfaces to a day or two of warm sunshine. Perhaps artificial baking of the clean, oiled forms at a moderate temperature would produce the same results. The obscure oil conditioning and saturation of the metal which presumably takes place (or the change in character of the oil) under such treatment will usually prohibit sticking for some time if the forms are properly cared for.

Although this is admittedly no explanation of some of the most mysterious cases of sticking of concrete to steel forms (and the equally mysterious stopping of the sticking), the above suggestion has been found to work, and it is hoped it may be found sufficiently feasible in some cases to solve the problem for many of the operators having this difficulty. LEWIS H. TUTHILL*

*Denver.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Studies on the system lime-ferric oxide-silica

MILTON D. BURDICK, Research Paper RP 1340—National Bureau of Standards, Vol. 25, No. 4, Oct. 1940. (Author's Abstract)

The equilibrium diagram of a part of the ternary system lime-ferric oxide-silica is presented. A comparison with the diagram of the lime-alumina-silica system shows that the presence of ferric oxide results in extension of the stability fields of the calcium silicates and the lowering of the liquidus and eutectic temperatures in much of the area investigated. Evidence from thermal analyses is offered of a limited solid-solution series between dicalcium silicate and ferric oxide, substantiated by an X-ray investigation. The amount of reduction of ferric oxide at the liquidus temperature of representative ternary mixtures is recorded. The applications to iron-rich cements are discussed.

Method for determining the moisture condition in hardened concrete

G. RUPERT GAUSE and JOHN TUCKER, JR., Research Paper RP 1334—National Bureau of Standards, Vol. 25, No. 4, Oct. 1940. (Authors' Abstract)

A method for studying the moisture conditions inside hardened concrete and cement paste is described. An electric hygrometer developed by the National Bureau of Standards was used to measure the relative humidity in cavities at various locations in the material. The relative humidity inside 12-inch concrete cubes stored at 50 per cent relative humidity for 300 days was found to range from 50 per cent near the surface to 66 per cent near the center of the cube. For cubes stored at 20-percent relative humidity the corresponding range was from 20 to 60 per cent. The relative humidity over cement paste of approximately normal consistency and cured in hermetically sealed bottles for 28 days was found to be 88 per cent for a moderate heat-of-hardening cement and 79 per cent for a high-early-strength cement.

New British standard specifications for ordinary and rapid hardening portland cements

Cement and Lime Manufacture, Vol. 13, No. 10, Oct. 1940, pp. 171-172. Reviewed by J. C. PEARSON

Specifications for portland cement, last issued in 1931, have been revised, and requirements for rapid hardening cement have been included in the revision. The requirements, without details, are very briefly summarized herewith:

Fineness: 100 g. samples sifted continuously 15 min. on the Standard No. 170 sieve shall have residues of not more than 10 percent for ordinary cement, not more than 5 percent for rapid hardening cement.

Chemical Composition: Percentage of CaO, after deduction for that necessary to combine with SO_3, shall not be more than 2.8 times per cent of SiO_2 plus 1.2 times per cent of Al_2O_3, plus 0.65 times per cent of Fe_2O_3, nor be less than two-thirds of that amount. Ratio of per cent of Fe_2O_3 to that of Al_2O_3 shall not exceed 1.5. Maximum permissible percentages as follows: Insoluble residue 1.0, MgO 4.0, SO_3 2.75, loss 3 in temperate climates, 4 in hot climates.

Tensile Strength: Standard mortars, 1:3, for ordinary cement 300 p.s.i. at 3 days, 375 p.s.i. at 7 days, and an increase over the 3 day strength. For rapid hardening cement, 300 p.s.i. at 1 day, 450 p.s.i. at 3 days, and an increase over the 1 day strength.

Compressive Strength: This is optional by the purchaser, in lieu of tensile strength. Cubes must have sides of 50 sq. cm. area; each cube mixed with 185 g. cement, 555 g. standard sand and 74 g. water, to be vibrated in a machine described in the specification. With load applied at 5000 p.s.i. per minute, minimum strength shall be for ordinary cement, 1600 p.s.i. at 3 days, 2500 p.s.i. at 7 days, and an increase over the 3 day strength; for rapid hardening cement, 1600 p.s.i. at 1 day, 3500 p.s.i. at 3 days, and an increase over the 1 day strength.

Effect of admixtures on temperature rise and time of set of cement pastes

Technical News Bulletin, National Bureau of Standards, No. 282, Oct. 1940.
Highway Research Abstracts

The effects of different admixtures on the rate of evolution of heat and on the hardening of three cement clinker pastes were recently determined by C. H. Jumper and George L. Kalusek. Temperature changes occurring during 24 or more hours were measured, and the time of set was observed. Aluminum, alkali, and calcium salts, a group of miscellaneous inorganic compounds, and a number of organic compounds were incorporated in varying amounts in the three cement clinker pastes. The changes in thermal effects and times of set were compared with those obtained for untreated clinker-water mixes. The changes induced in the early temperature rise (arbitrarily chosen at 1-hr.) differed among the three clinkers; but the effects on the main reaction; as manifested by the maximum temperature rise and the time to attain this rise, although varying in degree, were similar for the three clinkers. However, with the addition of different compounds to a given clinker, pronounced differences, even within a group of similar salts, were observed. Aluminum and alkali sulfates increased the rates of temperature rise, and in many instances caused very early initial set. Calcium salts induced a variety of changes in the times of set and in the thermal effects. Calcium acetate was notably a retarder of set. Copper chloride ($CuCl_2$), manganese chloride ($MnSO_4$), magnesium fluosilicate ($MgSiF_6$), and hydrofluosilic acid (H_2SiF_6) markedly increased the 1-hr. temperature rise and delayed the final set. Triethanolamine and its salts were vigorous accelerators of the set and the thermal changes. Some of the admixtures added to cements indicated that the presence of gypsum may, or may not, alter the effect of the admixtures, but no general correlation between the temperature rises and times of set could be deduced.

Freezing stability of highly porous material

N. ROTEN, *Teknisk Tidskrift*, No. 43, Oct. 26, 1940. Reviewed by BENGT. FRIBERG

The paper discusses the relation of length change behavior to stability at low

temperatures for highly porous, cementitious materials as used for insulating purposes. Materials which have successfully passed standard freezing and thawing tests have not always in wet condition proved resistant to low temperatures. Critical points have been window ledges and the outer surface generally in case of condensation at insufficient insulation and high inside humidity.

A cementitious material saturated by absorption contains the water in capillaries while the larger voids remain empty. During freezing fine ice needles are pressed from the capillaries into the voids. A primary requirement for freezing resistance is that the voids after absorption at least equal the volume required for the ice formation; the water absorption should be at the most 85 per cent of the total voids. Even for favorable void distribution, freezing stability is lacking if during the extrusion of the ice needles tension failures occur in the void boundaries; the material should not only have high tension resistance, but also high extensibility.

The presence of such conditions may be easiest explored by length change measurements of the freezing and thawing specimens. Four typical materials graphed for 2x3x8 in. specimens gave radically varying results. Exposed to — 30°C. in saturated conditions, all materials shortened during the first 15 minutes. The contraction in some instances was greater than that corresponding to the temperature decrease for the concrete material because of the much greater thermal coefficient of ice which filled the capillaries. The more permanent materials continued shortening to a minimum length; others, at some point thereafter assumed a very noticeable expansion of from .03 to .1 per cent.

To be exact, the observation of length change should cover the complete cycle of saturation, freezing, thawing, and drying to the original moisture content before saturation. Increase in length for the compiete cycle would indicate failures in the material and a questionable freezing stability.

Concrete mixing and placing on large dams: I, Organization and equipment

ADOLPH J. ACKERMAN, *Civil Engineering*, Vol. 10, No. 12, Dec. 1940, pp. 761-763.
Reviewed by J. R. SHANK

A proper construction plant is the most important step in building a large dam and its selection requires a thorough analysis to obtain proper correlation of time schedule, labor rates, river handling, topography and other local controlling factors. There is usually one best rate of construction for any one job and either faster or slower rates cost more. However, a faster rate may often be justified with its increased cost when earlier income for the project is obtained. The construction plant is usually the biggest single investment on the job and care should be taken that it is not under size. It is better to overplant than to underplant and thus be able to take advantage of breaks. The plant and placing system depends on (1) general type of dam structure, (2) limitations in speed of construction due to type of structure, (3) required completion date, (4) topography, (5) river handling and climatic conditions, and (6) economic rate of supply of aggregates. First cost should not stand in the way of designing the proper distributing system.

Though local natural sands and gravels of proper quality are generally the most economical, it is, however, feasible to use crushed rock for coarse aggregate and manufactured sand from this same source for fine. A typical cost for natural materials is 50 cents per ton. Crushed rock costs from 50 to 60 cents and manufactured sand $1.00 a ton. Large storage piles for several days of concreting should be provided. Cement is customarily handled in bulk by machinery. This handling costs

from 9 to 13 cents per barrel. Trucking of cement from the railroad to the job is economically feasible up to 15 miles.

In designing a mixing plant a detailed cycle in minutes and seconds of batching, mixing, transporting and placing of concrete must be developed to define the interlocking operations of each component of the plant. The capacity of the mixing plant should be about 20 to 25 per cent above the average rate of expected output. A full automatic batching plant is the most efficient for the larger plants. Mixing costs about 25 to 35 cents per cu. yd.

The most common type of form has been a pre-fabricated panel with horizontal anchorages at the lower edge and inclined internal struts and tie rods supporting the upper edge and anchored to the previously placed concrete on an angle of about 45°.

Twenty-five tons capacity cableways with 8 cu. yd. buckets have been used on free spans up to 2600 ft. The whirler crane with steel trestle runways across the site are best adopted to wide river dams, navigation locks, etc. The interrelation of whirler cranes, transfer and placing crews requires detailed study. For best performance the plant must operate at full capacity for as long a period as possible, and the inevitable interferences of other operations must be coordinated into the work with least possible adverse effect on the continuity of the work.

Concrete is almost universally vibrated into place with immersion-type vibrators. The concrete usually has a slump of from ½ to 1 inch, contains 0.85 of a barrel of cement per cu. yd. and attains a strength of 3000 p.s.i. for 28 days. Heat of hydration is controlled by embedded refrigeration lines.

Heavy mass concrete in dams cost about $5.50 per cu. yd. including all materials, plant, and labor. Low dams of massive type and lock structures cost $8.00 to $9.00 per cu. yd. Power-house substructures highly reinforced and with special forms cost $15.00 to $22.50 and super structures with a great amount of detailed form work cost $50.00 to $60.00 per cu. yd.

Sawdust concrete test results

L. W. NEUBAUER (University of Minnesota), *Agriculture Engineering*, Sept. 1940, Vol. 21, No. 9.
From the paper

"The object of the experiments discussed in this paper was to determine scientifically some of the critical characteristics of a concrete made of cement, sawdust, and water. Sawdust is known to be of organic nature and therefore undesirable in concrete because of its effect of slowing up or preventing the setting of portland cement. It has generally been considered unreliable as a constituent in mortar or concrete. A recent popularity and interest in its use has been in evidence, however, and this study was made in an effort to determine more facts concerning its strength and utility. Experimenters in other states (New Hampshire, Oregon, Pennsylvania, and Indiana) have worked principally with floors in poultry houses and hog and dairy barns. At Minnesota the first experiment was made by placing a sawdust-concrete floor in a hog barn with such unfavorable results that the technical laboratory studies were considered of great value."

"A total of 698 test cylinders of sawdust-concrete were made in the agricultural engineering laboratory, using 14 samples of wood of 11 different species. These cylinders were principally 2 in. in diameter by 4 in. in length, proportioned, mixed, placed, and cured in accordance with standard methods. A number of variables such as kind of wood, size of particles, proportions, water-cement ratio, age, and method of curing, were observed and recorded."

"The well-known effect of organic matter on the setting of portland cement and the influence of saturating and washing sawdust as already mentioned led to a study of soluble organic content in sawdusts. Nine varieties were placed in distilled water for eleven days, the liquor removed, water evaporated, and organic extracts determined. These values were plotted against the compressive strengths of the sawdust-concretes." * * * "A definite relation is evident. The woods represented are: (1) birch, (2) maple, (3) red cedar, (4) aspen, (5) jack pine, (6) cottonwood, (7) Norway pine, (8) ponderosa pine, and (9) spruce. Only points (1) and (8) are not closely correlated. The organic matter consists largely of gums, sugar, starch, and tannin. Red oak not tested for organic extractives contains considerable tannic acid and forms poor concrete. The point would fall along the lower end of the curve. There also would lie the point for Douglas fir with its zero strength and well-known high organic content."

CONCLUSIONS

"I. Cement-sawdust concrete is rather uncertain and unreliable and its strength cannot be easily predicted.

"2. Its weakness is generally due to soluble organic content which interferes with the setting of cement, and wood of high soluble organic content should be avoided.

"3. The water-cement ratio is especially important and should be watched with extreme care.

"4. This problem may be simplified by previous saturation of the sawdust in water.

"5. Mixing proportions are preferably about 1:3 or 1:3½. Richer mixes are strong, heavy, conductive, and expensive. Leaner mixes are cheap, weak, nondurable, and inflammable.

"6. Best utility is secured where strength is not vitally important but where economy, light weight, insulating value, nail-holding ability, and resistance to burning and termites are desired.

"7. It is always advisable to test a small trial batch for a one or two-day period before mixing any kind of saw-dust in quantity.

"8. Some recommended species of sawdust are spruce, Norway pine, jack pine, and aspen.

"9. Woods to be avoided include cottonwood, oak, Douglas fir, birch, maple, and red cedar."

Common salt for ice removal on concrete pavements

A. R. COLLINS, Road Research Laboratory (England), *Roads and Road Construction*, p. 98, May 1, 1940. Highway Research Abstracts

A common method of removing thin layers of ice or frozen snow from the surface of concrete pavements is through the application of common salt. In certain circumstances this treatment may have deleterious effects on the concrete, usually consisting of scaling or spalling of the surface. This damage is not always apparent until some time after the application. Although a number of instances of this damage had been brought to the attention of the Road Research Laboratory, it was not possible to obtain information as to the quality of the concrete, the rate of application of the salt, the prevailing temperatures, and other of the important factors involved. It was therefore decided to conduct in the laboratory under controlled conditions, a series of tests to examine the general characteristics of the resulting damage and to determine the most suitable method of application to minimize or avoid such damage.

The test specimens used were 18 in. square slabs, 2½ in. thick, with a raised edge so that water could be retained on the surface. Three types of aggregates were tested with a normal portland cement, in 1:2:4 proportions. Varying water-cement ratios were used, and the slabs tested at varying ages. The other variable was the rate of application of the salt.

In performing the tests, the surface of the slab was covered to a depth of about ⅛ in. with water and then frozen at −10°C. for 16 hours. The salt treatment was then applied, and the temperature allowed to remain at −10 for another two hours. The specimens were then brought into room temperature and allowed to thaw for six hours, after which they were washed and the cycle repeated. The salt used was commercial common salt (sodium chloride), but was crushed to a fine powder in order to obtain a more even distribution when mixed with the sand. Tests with straight salt were performed without this crushing.

The table on the following page shows the effect of 5 treatments on the rate of disintegration of frozen concrete which was mixed with a w/c ratio of 0.55. All samples were subjected to 40 cycles of the treatment.

It was noted that applications of a 1:50 salt-sand mixture was not sufficient to thaw any but the thinnest ice films, although it did permit the sand to become embedded, thus giving a roughened and less dangerous surface. It was noted that the effect of the age of the specimen was more noticeable with the higher rates of salt application. A comparison of the resulting surface on mixes of different water-cement ratios showed considerable variation in the amount of damage, indicating that the quality of the concrete plays a large part in the failure. Little apparent difference could be noted with the different aggregates used.

Type of treatment	Age of concrete	Number of cycles required to bring about specified degree of failure			
		First signs of flaking	Slight flaking	Pronounced flaking	Loosening of large aggregate
1	14 days	1	3	10	20 to 30
	28 days	2	5	15	30
	3 months	2	10	30	..
2	14 days	1	3 to 5	30	..
	28 days	2	7 to 10	35	..
	3 months	3	20	..	..
3	14 days	2	5	30	..
	28 days	2	5 to 10	35	..
	3 months	3	20	..	..
4	14 days	3	5 to 10	..	..
	28 days	3	10	..	..
	3 months	4	25	..	..
5	14 days	5	10	..	..
	28 days	5	..	..	..
	3 months	5	..	..	..

1. 1½ lb. salt per sq. yd. 2. ¼ lb. salt per sq. yd. 3. 1:8 salt-sand mixture, 2 lb. per sq. yd. 4. 1:50 salt-sand mixture, 2 lb. per sq. yd. 5. No treatment.

Also investigated was the effect of heating the application before applying. The sand-salt mixture was more effective when heated than when cold, but only for the short time before it reached the surrounding temperature; afterwards the rates of thawing were sensibly the same for both the hot and cold applications.

As a result of the experiments, the following tentative recommendations for the use of salt for ice removal were suggested:

1. Salt should never be used alone because of the danger of high local concentrations.
2. For removal of thick ice layers, a mixture of 1 part salt to 8 parts sand by weight should be applied at the rate of 2 lb. per sq. yd., and then only in particularly bad cases.
3. For removal of thin films, a mix of 1:25 should be used at the 2 lb. per sq. yd. rate. This is suitable for general application.
4. If it is possible, the mixture should be heated to a temperature of 300° to 400°F immediately before application, as the immediate effectiveness is increased.
5. It is important to mix the materials thoroughly. This may be accomplished by adding the salt to the sand in the form of a solution in water.
6. Fine ashes or other inert hard material may be used in place of sand, but if the density differs materially from that of sand, the proportions by weight should be adjusted accordingly.

An Electrical resistance method for making continuous measurement of moisture in concrete pavements and in soils under road conditions

George Bouyoucos, Michigan Agricultural Experiment Station, East Lansing, Michigan.

Highway Research Abstracts

In previous communications (1) (2), an electrical resistance method was presented for making a continuous measurement of soil moisture under field conditions. The method made use of a Plaster of Paris block in which were imbedded two electrodes, and a special form of the Wheatstone bridge. The principle of the method was based upon the following facts:

(1) The Plaster of Paris block takes up and gives up water very readily, so that it tends to maintain a close moisture equilibrium with its environment.

(2) The electrical resistance of the Plaster of Paris block is inversely proportional to its water content, so that by calibration, the electrical resistance becomes an index of moisture in soils or in concrete.

(3) The comparatively high solubility of Plaster of Paris, which is 2200-2400 ppm, tends to act as a buffer and minimizes the influences of salts.

(4) By imbedding the electrodes in a uniform and constant material such as Plaster of Paris, the uncertainties and errors which arise from differences in texture, structure, composition, compaction, salt content, etc., of the medium surrounding the electrodes, are eliminated or greatly minimized, and the results thereby become more reliable and accurate.

The method as applied to soil has proven satisfactory. In the present note, it is the purpose to call attention to the possible application of the method to the measurement of moisture in concrete pavements under field conditions, and to other studies connected with concrete and soils.

It has been found that the method is capable of making a continuous measurement of moisture in concrete from the saturation point down to the air dry condition, as shown in the table.

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(1) *Science*, Vol. 89, March 17, page 252, 1939.
(2) *Mich. Tech. Bull.*, 172, 1940.

RELATIONSHIP BETWEEN ELECTRICAL RESISTANCE AND MOISTURE CONTENT IN CONCRETE:

Resistance at 70° F. Ohms	Moisture Per Cent	
285	10.3	Saturated
355	9.3	
2,500	8.7	
5,800	7.3	
18,000	6.5	
38,000	5.5	
75,000	5.1	
200,000	3.9	
1,000,000	3.3	Air dry

These results were obtained in the laboratory by casting around the Plaster of Paris block, a shell of mortar ¼ in. thick. This mortar was made up by mixing in the following proportions by weight: 50 parts of water, 100 parts Huron cement, and 200 parts sand, passing 2 mm. sieve. After the mortar had been properly cured, it was allowed to dry at room temperature to different degrees of dryness, as indicated by the electrical resistance readings.

In order that the mortar shell might have a uniform moisture throughout its thickness, the entire block was wrapped in oiled paper and allowed to stand for twenty-four hours or longer. Then the resistance reading was taken, the mortar shell broken off from the Plaster of Paris block, and its moisture determined by drying it at 100°C. for twenty-four hours.

Other cells, besides Plaster of Paris cells have been tried with the method, but the Plaster of Paris cells have proven to be the most satisfactory. Other cells tried were fired clay, builder's cement, various concretes, cement and Plaster of Paris, marble dust with various binders, dental casting compounds, Keene's cement, and various commercial plasters (Alabatine, Alabastone, etc.)

Air dry condition seems to be the lowest degree of moisture that the method is capable of measuring. The moisture content at the air dry condition may vary somewhat, depending upon the variation in temperature and humidity, as well as upon composition of the mortar.

It would seem that a part of the moisture contained in concrete in the air dry condition is water which is probably held with great chemical or physical forces and can only be determined by expelling it at high temperatures.

In order to measure such low moisture contents in concrete and in soils, a special electrical bridge has been developed which can be used to read very high resistances. This bridge is portable, and weighs about fifteen pounds. A vacuum tube oscillator, powered by dry cell batteries, supplies alternating current at high frequency, eliminating polarization and electrolysis errors. The capacitance introduced by the Plaster of Paris block is balanced out by a variable condenser.

Besides measuring moisture, this electrical resistance method can also be used to ascertain whether or not the concrete or the soil is frozen. It has been found that when the water freezes the resistance jumps from about 500 ohms to about 50,000 ohms.

From all tests thus far made it seems that the method is satisfactory in making a continuous moisture measurement of soils and of concrete pavements at any depth under field conditions.

A detailed report of laboratory and field study of the method will be made by the Research Division of the Michigan Highway Department.

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Flat Plate Rigid Frame Design of Low Cost Housing Projects in Newark and Atlantic City, N. J.*

By Joseph DiStasio†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Six low-cost housing developments in Newark and one in Atlantic City, either recently completed or under construction, are described. Economies in cost and speed of erection were effected by the use of a novel type flat plate rigid frame design. Typical details are shown, and a summary discussion given of the basis of design, construction methods, and comparative costs.

INTRODUCTION

In recently completed low cost housing developments in Newark and Atlantic City, a novel type of flat plate rigid frame design has been employed. In what follows, the projects will be described, design procedure outlined, details of construction recounted, and items affecting costs analyzed. It is believed that experience gained on these structures may prove of interest to those engaged with similar work in other sections of the country.

DESCRIPTION

The Newark, N. J., low cost housing development consisted of six projects. Five of these were reinforced concrete frame buildings of three upper stories and part basements, and the sixth was two-story non-fireproof with reinforced concrete floating or pile foundations. All the buildings were composed of combinations of variously shaped typical units arranged in different ways to form complete buildings. Units were either straight, L-shaped, or T-shaped in plan and each

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†J. DiStasio & Co., Consulting Engineers, New York, N. Y.

included several apartments on each floor. They were combined to suit the shape of the plot and to provide variety of architectural appearance. Boiler house, administration, and recreation units were included in each project either as separate structures or attached to one of the buildings. All sites were completely landscaped with parking areas, drying areas, spray pools, play grounds, etc. The various projects are summarized in Table 1.

TABLE 1—BUILDINGS OF THE NEWARK HOUSING DEVELOPMENT

Name and Location Newark, N. J.	No. Bldgs.	Total Apartments or D. U.	Total Rooms	Sq. Ft. Supp. R. C. Floors and Roof
Pennington Court, South & Pacific Sts.....	4	236	984½	228,000
Seth Boyden Court, Dreamland Park.....	12	530	2241	494,000
J. M. Baxter Terrace, Orange & Nesbitt Sts.	21 & BH	621	2638½	585,000
Felix Fuld Court, Livingston St.....	8 & BH	300	1314	275,000
J. W. Hyatt Court, Roanoke St.....	12 & BH	402	1746	373,000
Stephen Crane Village, Branchbrook Park..	27 & BH	354	1494	Non-fireproof construction

At Atlantic City, the low cost housing project known as The Jonathan Pitney Village contained 21 two-story non-fireproofed buildings with reinforced concrete first floor, grade beams, and foundations, six three-story reinforced concrete frame buildings with supported first floor, one reinforced concrete frame building with three upper stories and basement, and a reinforced concrete boiler and community house. These buildings were all composed of various straight units of varying length, and contained 333 apartments and 1431½ rooms. The total supported floor area was 203,000 sq. ft. reinforced concrete floors and roof, and 175,000 sq. ft. non-fireproof construction. Landscaping, parking, and playgrounds were also provided.

The buildings generally were 27 ft. wide out to out, and the structural frame consisted of a 5½-in. flat slab without beams, except at stairways and spandrels, supported on three lines of columns. To suit the architecture, in the earlier projects, the center row of columns was placed 10½ in. off the center line of the building, but in later projects the architectural layout was modified to provide two equal spans of 12 ft. 7 in. each. In the long direction, the columns were spaced at random spans to suit the room layout with a maximum of 16 ft. 6 in. A unique feature was provided by making the spandrel beams of sufficient strength so that wall columns need not line up with the interior columns. In view of the light loads involved, this was accomplished at no extra cost except for increasing the spandrel reinforcing in a few of the longer spans as minimum steel requirements were usually sufficient. With no interior beams and flexible column locations, the architects had complete freedom in establish-

ing their room layouts with the minimum clear story height of 8 ft. (See Fig. 2.)

Spandrel beams were 9½ in. by 17 in., stair well beams 8 in. by 16 in., and landing beams 10 in. by 8 in. All beams were reinforced with a minimum of two ⅝-in. round rods top and bottom, and provided with stirrups throughout to resist torsional stresses. Interior columns were 15 in. by 15 in. or 14 in. by 14 in. with a 5½-in. 45-degree splayed head. Wall and stair columns were 13 in. by 13 in. or 12½ in. by 12½ in. without capitals. All columns were reinforced with a minimum of four ¾-in. round bars and ¼-in. round ties 11-in. on center. Vertical bars were spliced 24 diameters at each floor and hooked at the top into the roof slab for anchorage. Basement walls were carried on reinforced concrete grade beams between footings. These grade beams also acted as basement wall and water table to a point just above grade which varied for the different buildings. Entrance porches and platforms were cantilevered out from the first floor stairwell beams and columns. All columns started on pedestals and concentric spread footings or pile caps as determined by the soil conditions at the various sites.

Exterior walls generally were composed of 4-in. face brick backed up by 6-in. cinder block and finished on the inside with an air space and plaster furring on metal lath. Vertical slots were provided in all wall columns for brick ties, and loose steel angle lintels spanned over doors and windows. Partitions within apartments were 2-in. plaster on metal lath, and those between apartments consisted of 3-in. terra cotta plastered both sides except in stair halls where glazed tile was used. Due to the use of plywood forms, no plaster was used on the ceilings, which only required removal of fins to be ready for the finished painting. Roof drainage was secured in some cases by screeded cinder concrete fill, and in others by pitching the top of the roof slab 1 in. to drains, but soffit of slab was kept level.

Although some buildings had straight lengths of as much as 230 ft., no expansion joints were used in these projects. Studies made of existing structures indicated the generally unsatisfactory behavior of such joints. It was considered preferable to minimize any small cracking by careful design of the reinforcement with due regard to the shapes of the buildings. Where the cross section of a building was reduced by stairwell openings, additional top and bottom reinforcing was provided in the slabs. Special top and bottom and diagonal bars were used around all incinerator and duct holes. Continuous top and bottom bars were used in all spandrel beams, and particular attention was given to anchorage of column rods in the roof slab.

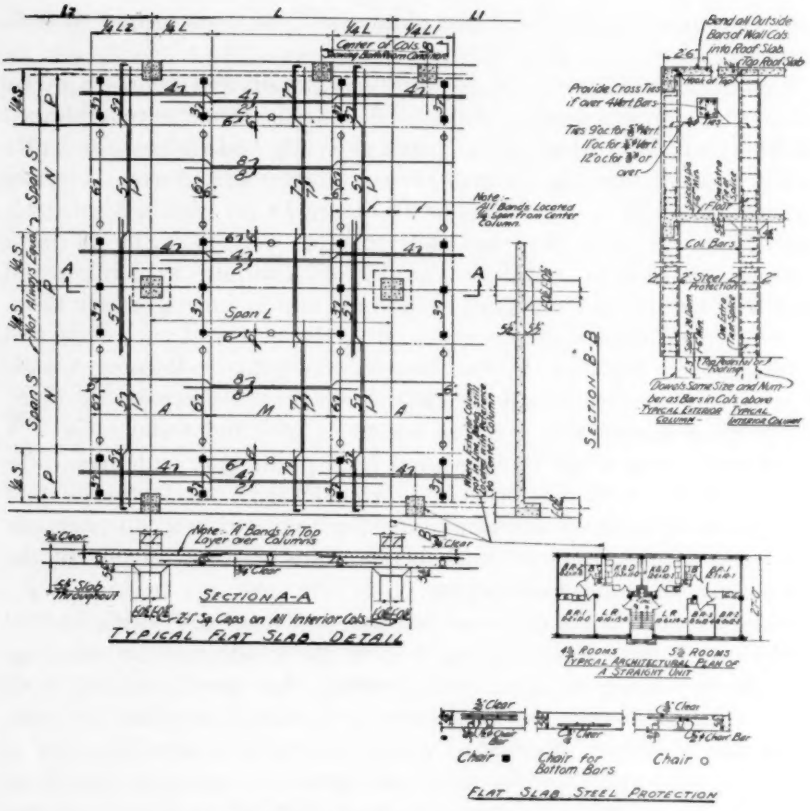
Span S
Span L
Span S
Note "M" Bands in Top Layer over Columns
SECTION A-A
SECTION B-B
TYPICAL FLAT SLAB DETAIL
Bend all Outside Bars of Wall Cols. into Roof Slab
Cap Roof Slab
Provide Cross Ties if over 4 Vert Bars
Ties 9 oc for ⅝" Vert
11 oc for ¾" Vert
12 oc for ⅞" or over
Col. Bars
Steel Protection
Dowels Same Size and Number as Bars in Col's. above
TYPICAL EXTERIOR COLUMN
TYPICAL INTERIOR COLUMN
4½ ROOMS
5½ ROOMS
TYPICAL ARCHITECTURAL PLAN OF A STRAIGHT UNIT
Chair ■
Chair for Bottom Bars
Chair ○
FLAT SLAB STEEL PROTECTION

FIG. 1A—(SEE FIG. 1B, OPPOSITE)

Directions for Placing Flat Slab Reinforcement

Lines separating bands shall be located ¼ of the panel width from the column centers.

Bands running short way of building shall be based on the center column.

Bands running long way of building are generally designated P, T, or N.

Bands running short way of building are generally designated "A" or "M."

All bars shall be placed in the following order:

1. Place straight bottom bars of "A" bands supported on ¾-in. high cement or metal chairs.
2. Place straight bottom bars of "P" bands supported on ¾-in. high cement or metal chairs.
3. Place ½ϕ chair bars resting on 4-in. by 4-in. cement or metal chairs, ■, 3⅛-in. high, 6 in. inside band lines.
4. Place bent bars of "P" bands and top bars of "T" bands resting on these chair bars. Between bends in bottom of slab, wire 3⅜-in. ϕ tie bars to straight and bent bars of "P" bands.
5. Place bent and top bars of "A" bands resting on "P" and "T" bars just placed. Between bends in bottom of slab, wire three ⅜-in. ϕ tie bars to straight and bent bars of "A" bands.
6. Place ½-in. ϕ chair bars resting on 4-in. by 4-in. cement or metal chairs, °, 3⅞-in. high, 6-in. inside band lines.
7. Place straight and bent bars of "M" bands bent up bars resting on chair bars and elsewhere bars supported on ¾-in. high cement or metal chairs.
8. Place straight and bent bars of "N" bands bent up bars resting on chair bars and elsewhere bars resting on "M" band.
9. Wire bars at all intersections.
10. All bent and top bars to have 8 in. diameter right angle hooks at end supports.

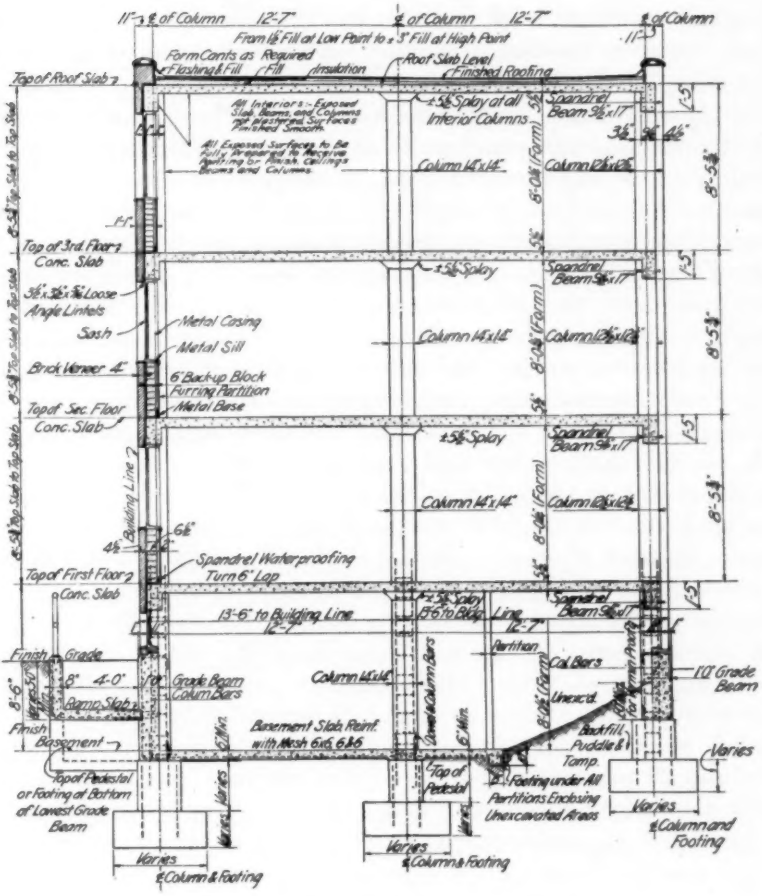

FIG. 1B—(SEE FIG. 1A OPPOSITE)

DESIGN

Design throughout was based on A. C. I. stresses for a concrete having an ultimate strength of 2500 p.s.i. at 28 days. Analysis of the structure was made by considering the slabs and columns to form a rigid frame using the Fixed Point Method described in "Structural Bending Moments Simplified" by U. T. Berg. For practical reasons it was advisable to maintain the same slab thickness and column sizes throughout. As preliminary considerations, limiting spans were determined by the usual flat slab formulas for slab thickness, and diagonal tension at depth out from splayed column heads, and bending stresses were checked after maximum moments were determined.

Diagonal tension at a wall column was computed on a section at the depth out from the face of column for a width equal to the column plus twice the depth of the spandrel beam.

In the rigid frame design, shape coefficients were first determined for the columns and panel width of slab considering the joints and capitals to be rigid with infinite moment of inertia within these regions. Far ends of columns one story above or below the floor being investigated were considered fixed, although a typical check calculation was made considering the complete four-story frame from foundation to roof as one unit. For all practical purposes, the assumption of far ends fixed, was deemed of sufficient accuracy, and this method was applied throughout. Slab distribution and carry-over coefficients were next determined, and final total width moments at the column center lines determined by applying the most unfavorable combinations of live and dead load to the various spans. In the short way of the building, calculations were based on the actual two span condition, and in the long way, on the assumption of an infinite number of spans corrected both for end spans and for variations in length of adjacent spans. Slab negative moments at interior columns were reduced to a point at one half the slab depth out from the column splay, and at wall columns to the face of the column. Following A. C. I. Code (1936), negative moments were distributed 74 per cent to the column band and 26 per cent to the inner band for interior columns. At exterior supports, the distribution was 80 per cent and 20 per cent except that extra top bars were added in the inner band along the wall to equal those between interior supports to insure adequate slab strength and develop the torsional resistance of the spandrel beams where exterior and interior columns were not in line.

In view of the uncertainty of the assumption of infinite moment of inertia of the splayed column head, positive moments were determined by similar methods based on constant moments of inertia between center lines of supports for the column and for the slab. This gave somewhat higher positive moments than in the variable moment of inertia analysis. Thus, the structure was safeguarded for both positive and negative moment conditions. Positive moments were distributed 58 per cent to the column band and 42 per cent to the inner band. All band widths and locations were based on the center row of columns so that no skewed bands were required, thus insuring against more than two layers of crossing reinforcement in the slabs.

Similar calculations were made for the roof where only one story of columns contributed restraint. Because of the lighter roof load,

moments were generally less than for the floors except in short end spans where moments over first interior columns were considerably increased.

It was found that only one set of calculations was required for the two-span direction, as, in view of the stiffness of the columns compared to the slab, the moment per foot of panel width was practically constant for the range of panels used. In the long direction, about four calculations were generally made, and tables of reinforcement for the various spans made by interpolation. Resisting moments in columns were checked from the computed moments, and while considerable tension was developed in the top story wall columns, the minimum reinforcement of four ¾-in. round bars was found sufficient to keep the resulting stresses well within accepted limits.

Slab reinforcement consisted of straight and bent ⅜-in. and ½-in. round bars with some additional straight bars over the supports. Maximum spacing was limited to twice the depth of the slab or 11 in. For uniformity in placing, the same number of bars were used throughout in the inner bands of the long direction, area variations being adjusted by changes in relative number of each size bar. This permitted the continuous alignment of all straight and bent bars throughout the floor. Directions governing the order of placing slab bars were carefully worked out and rigidly followed to insure proper positioning of the reinforcing. Half-inch chair bars and cement block chairs of proper height were provided and slab bars were wired together at the intersections. All beam and slab bars were anchored by hooks at end supports and by suitable laps at intermediate columns determined and standardized from moment diagrams. (See Fig. 2.) Originally, standard semicircular hooks were specified, but due to difficulty in placing encountered in the field, old fashioned right angle hooks were adopted and appear to have given entirely adequate service. It is well to point out that the latest revision of the A. C. I. Code permits a right angled hook of increased length due to this practical consideration. The arrangement of the slab reinforcement in a typical floor of one building is shown in Fig. 3.

CONSTRUCTION

On the Newark projects, ⅝ in. thick plywood was used to line all form work of exposed surfaces. For the slabs, plywood was furnished in 4 ft. 0 in. by 8 ft. 0 in. sheets and was supported on 3 in. by 4 in. joists 14 in. on center. These joists were carried on 3 in. by 6 in. girders about 5 ft. 6 in. apart which in turn rested on 4 in. by 4 in. posts wedged up from a 2 in. by 6 in. floor plate. Column sides were formed

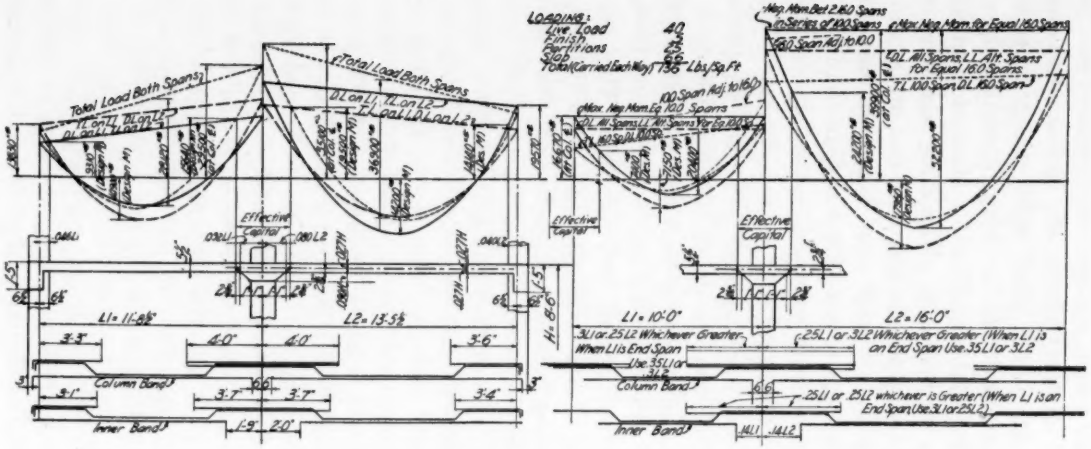

FIG. 2 (LEFT)—BENDING MOMENT DIAGRAM AND ARRANGEMENT OF REINFORCEMENT FOR TWO SPAN DIRECTION WHERE SPANS ARE UNEQUAL

Bending moments shown are per foot of total panel width based on a 14-ft. 0-in. wide panel. They were distributed between column and inner bands as explained in the text. Analysis for maximum negative moments was based on variable moments of inertia, and maximum positive moments on constant moments of inertia.

FIG. 2 (RIGHT)—BENDING MOMENT DIAGRAM AND ARRANGEMENT OF REINFORCEMENT FOR CONTINUOUS DIRECTION FOR EQUAL OR UNEQUAL SPANS

Bending moments shown are per foot of total panel width based on a 12-ft. 7-in. wide panel. The distribution between column and inner bands and the basis of analysis were similar to the two-span direction.

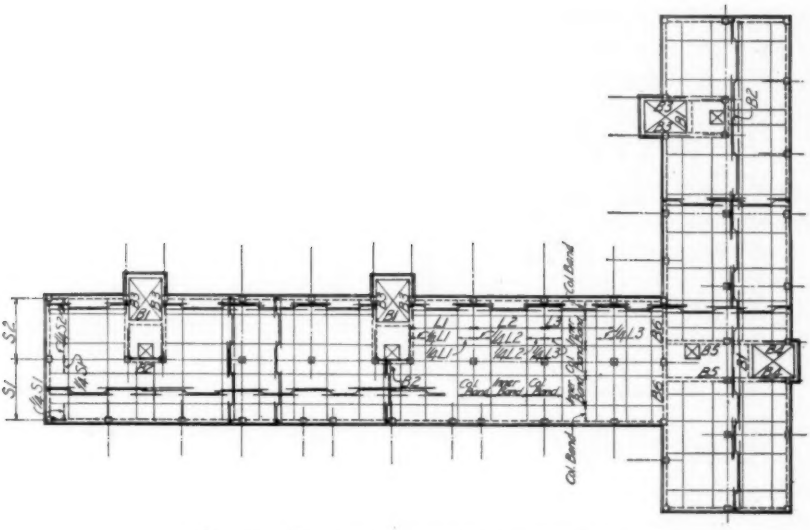

FIG. 3—TYPICAL FLOOR FRAMING PLAN

in single plywood sheets of required width stiffened at the edges by 1 in. by 6 in. battens, and the assembly spiked together with 1 in. by 4 in. cleats 15 in. apart. Interior column splays were formed completely in plywood. Beam forms were of similar type to the columns except that the 1-in. pine backing on the sides was solid to permit nailing on a 1 in. by 4 in. strip to support the slab centering. Soffits were supported on 4 in. by 4 in. T-jacks about 2 ft. 6 in. on center. Plywood was omitted on the outside face of spandrel beams which would later be faced with brick, and the outside form braced every 4 ft. 0 in. to a 2 in. by 6 in. runner resting on the T-jack arm extended. Posts were set under the outer edge of the spandrel beam to balance the kick of the outside form, and were braced across the building with 1 in. by 6 in. ties. (See Fig. 4.) Clear story heights were constant to permit reuse of forms under all floors. (See Fig. 1.)

Local labor union rules require all reinforcing bars to be bent on job and all lengths, of $\frac{5}{8}$-in. bars or less, under 20 ft. 0 in., to be cut on the job, also. Generally, rods were delivered to the site 6 days prior to the concreting date. This enabled the crews to cut, bend, assemble, and place steel, and permitted the mechanical trades to install their material prior to pouring. The plumber followed with roughing as soon as floor was stripped so that by the time the roof was reached, plumbing could be completed in very short order and the test made. This allowed the immediate starting of construction of

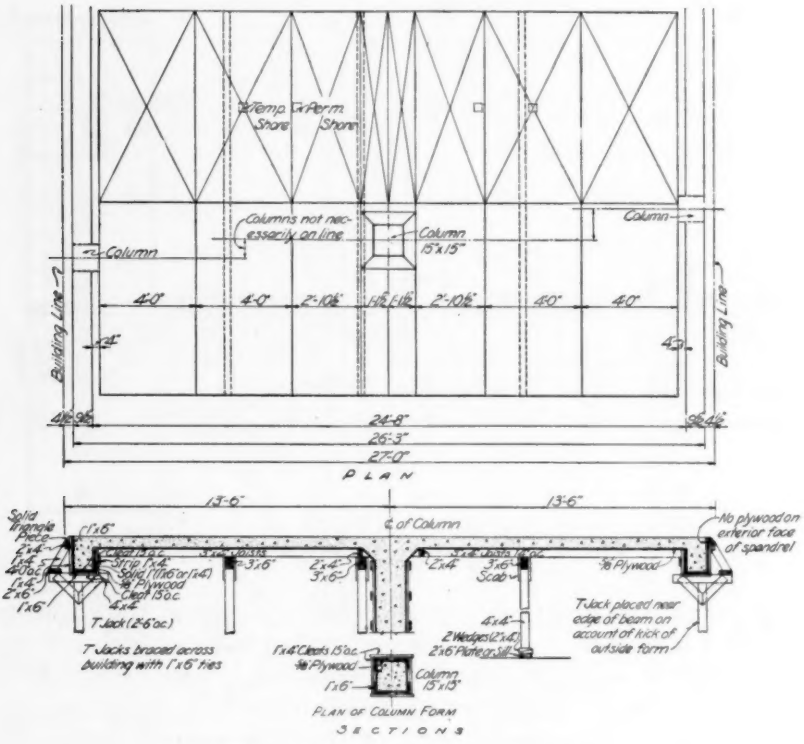

FIG. 4—TYPICAL FORM DETAILS

slabs on ground, partitions, and stairs. Brickwork followed the concreting and plumbing, so that there was never any interference between the various trades.

Concrete was delivered to the site in special mixer trucks. Under the supervision of an inspector, water was added, and the mixer drum turned over after arrival on the job. The mixer discharged into one cubic yard bucket which was raised to the placing elevation by means of a gasoline driven speed crane with an 85-ft boom. The concrete was dumped and spread by the use of a gasoline driven portable vibrator. Immediately thereafter, the slab was screeded and troweled to the finished floor surface. All flat surfaces were covered with two thicknesses of burlap and kept wet for 3 days.

From 5 to 8 days, depending on atmospheric conditions, after the concrete had been poured, temporary reshores were placed at 8 ft. 0 in. intervals and original shores and flat forms removed except the sheets immediately over the temporary reshores. Next, permanent

reshores were installed adjacent thereto to permit stripping of the remaining plywood. Thus the fresh concrete was supported at all times and safeguarded from construction load stresses before it had attained requisite strength. The permanent shores remained in place for 28 days. (See Fig. 4.)

Due to the use of internal vibrators and plywood forms, finished surfaces were generally smooth. In a few cases, some honeycombing due to segregation of aggregate was observed in column shafts and outside faces of spandrels. These were immediately pointed up following stripping. In general, the finished concrete was remarkably free from defects and required very little work other than removal of fins.

All concrete was specified to be 2500-lb. concrete. The contractor was given the option of furnishing a "Fixed Volume Mix" or a "Controlled Mix" concrete. For the fixed volume mix, it was specified that the total volume of fine and coarse aggregates measured damp and loose should not exceed 5½ times the volume of cement with a minimum of 5½ sacks of cement to the cubic yard. Control mix concrete was required to have at least 5 sacks of cement to the cubic yard and the contractor was to pay for all tests. These specifications more or less followed suggestions embodied in U. S. H. A. Master Specifications. In view of the additional costs of the tests, the contractors chose the option with "fixed volume" concrete as the extra cost of the ½ bag of cement was more than offset by the increased costs of tests and inspection. However, the local Housing Authority retained testing laboratories to check that the strength of the concrete and other requirements were in conformity with the specifications.

In spite of exposure to all types of weather and temperature, cracking of concrete has been nil. Due to the flexibility of the frame structure, there has been no cracking of brickwork especially at wall corners such as has been noticed throughout the country in wall bearing type buildings. No cracking has been observed in slabs or beams near stairwell openings indicating the effectiveness of the extra top and bottom reinforcing provided at marked changes in the cross section of the building in distributing temperature stresses. The fact that the slabs were 5½ in. thick provided adequate covering of electrical conduits and other piping, especially at the first supported floor level where clusters of piping may prove detrimental to thinner slabs such as occur in beam and slab structures. Again, due to the uniformity of having a slab of constant thickness without beams through-

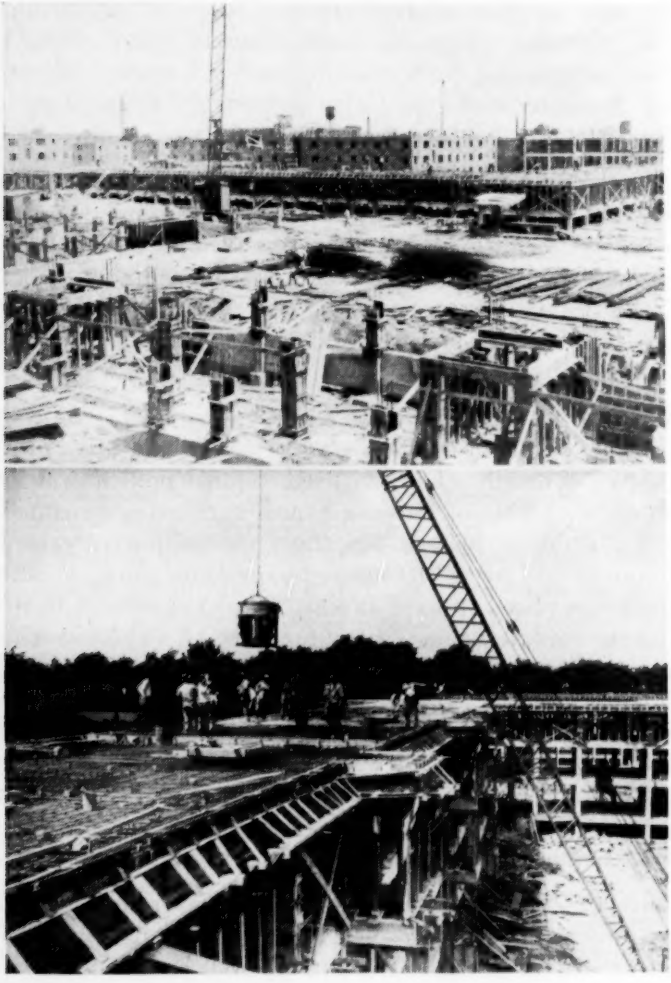

FIG. 5—SETH BOYDEN COURT UNDER CONSTRUCTION

out the floor, there is less tendency for temperature stresses to localize at weaker sections. Occasionally, however, in the roof finish, which was only screeded, hair line cracks have been noticed which, especially in hot weather, appeared almost immediately after pouring and before removal of forms. These are due to checking and shrinkage in cement. They have been very few considering the great amount of roof area poured to date, and have no structural significance after the roof covering is applied.

COSTS

The structural concrete quantities involved in these projects averaged as shown in the following table subject to some variation in the foundations due to different site conditions. Foundations given below are based on concentric spread footings at 2½ to 3 tons soil pressure.

Item	Quantity per Sq. Ft. of Supported Floor & Roof		
	Reinf. Steel Lbs.	2500 Lb. Concrete Cu. Yds.	Forms Contact Surface Sq. Ft.
Slabs, incl. chair bars	2.07	.017	1.00
Interior beams	.38	.001	.08
Spandrel beams	.62	.002	.20
Columns	.69	.004	.35
Foundations & Grade Beams	.68	.006	.30
Total	4.44	.030	1.93

Roof screed or floor finish 1.00 sq. ft.
Interior finish exposed surfaces 1.35 sq. ft.

On the last project, soon to go out for bids, we estimate that the reinforcing steel will run about 4.22 lbs. per sq. ft. These unit quantities of construction may be used, with proper cost units which apply in the region where the job is being figured, to give a comparative cost for the reinforced concrete frame.

Use of reinforced concrete skeleton frame construction permitted exterior walls of 4-in. face brick, 6-in. cinder block backing, and plaster furring on metal lath instead of solid masonry and furring required in a wall bearing job. In a typical story, spandrel beams and columns embedded in the wall replaced 6-in. cinder block and plaster furring on metal lath on 23 per cent of the gross exterior wall surface. For a net section deducting windows, this percentage will increase considerably. This direct saving of materials, in masonry, furring, and plaster, more than offsets the increased cost of the exterior framework, and, in analyzing comparative costs with brick bearing structures, the consideration of this factor is important. Due to the absence of beams, there was considerably less exposed concrete surface to be rubbed with consequently a lowering of the price of the work. Where cinder concrete roof grade and screed was omitted, and grading and finishing of the roof slabs substituted, roofing could be built and roofing started immediately. This saved about three to four days in the enclosing of each building.

Of far greater significance, both from an economic and from a construction standpoint, was the saving in time and equipment due to

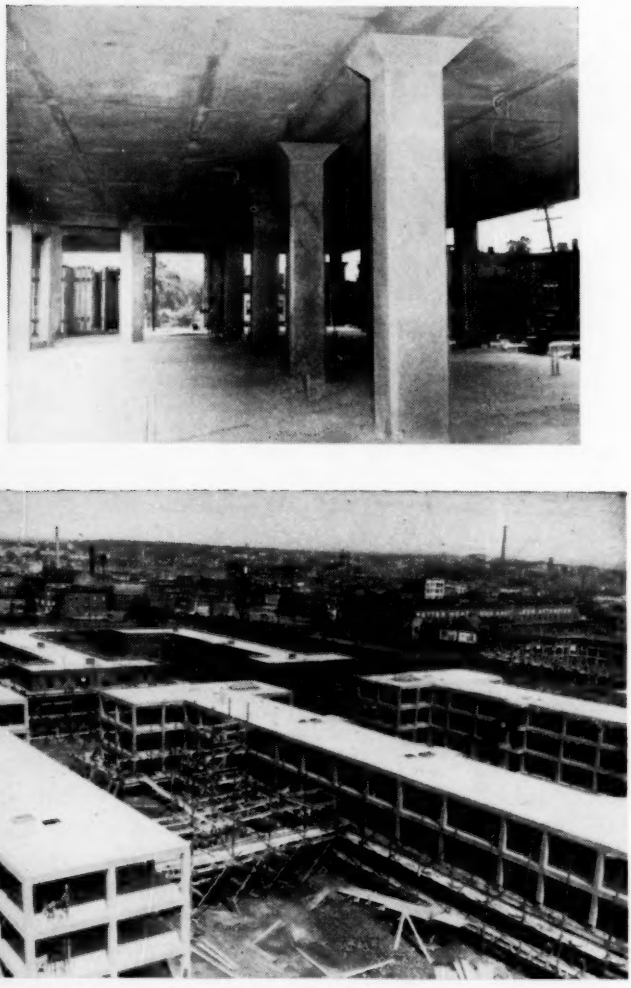

FIG. 6—INTERIOR AND CONSTRUCTION VIEWS OF JAMES M. BAXTER TERRACE

the skeleton frame design, permitting the various trades to follow each other without delay. Concrete work, plumbing, and masonry progressed continuously in regular order with no interference, and there was no waiting by the concrete contractor for the mason to build the bearing walls. This continuous operation of the respective trades also permits a considerable saving in hoists, chutes, etc., by the various trades, which would otherwise have to be either moved to a new location or duplicated in another part of the site, in order not to

delay their operations. This item of continuous operation has a great influence on contractor's overhead costs. For example, when one subcontractor is held up by strikes or delayed shipments, others can continue to complete their work within the specified time. The design of these buildings directly influences the time of completion. For instance, a group of 18 three-story buildings of wall bearing design in a nearby city built by the same concrete subcontractor required 14 months to complete the superstructure concrete and exterior brickwork. The foundations of this group had been installed prior to the letting of the contract for the superstructure work. In Newark, a group of 21 buildings, similar in layout, of skeleton flat plate rigid frame design, and having approximately 25 per cent more concrete, had all of the concrete for foundations and superstructure completed in 6 months with the brickwork following 4 weeks behind. Thus a skeleton frame design involves many savings not reflected in material quantities, but reflected in final total cost.

In Atlantic City, one square block of housing was of reinforced concrete flat plate with exterior brick facing and 6-in. cinder block backing similar to the Newark design. Comparative designs were made using a center row of columns and girders, and slabs bearing on exterior 12-in. brick bearing walls. Accurate cost estimates were made and priced by a local contractor having wide experience in building low cost housing. This cost estimate indicated a saving of about 5 cents per sq. ft. of supported floors and roof areas for the flat plate reinforced concrete frame design over the usual brick bearing design. This confirmed similar comparative cost estimates, obtained for the Newark area. To this apparent saving in cost of erection and materials, must be added the other more important savings in time, speed and efficiency of operation, discussed above, in detail.

As stated, in Newark there are six projects to date, the first being a group of four projects and the latter, two. Of the last two, J. W. Hyatt Court is now ready for bids, while the other, Felix Fuld Court, the contract is in the process of being awarded. Of the first four projects, Pennington Court is now completed, and the other three are in various stages of completion. The speed of construction, for the first project, Pennington Court, is as follows: ground was broken on May 23, 1939; the first units opened for inspection January 20, 1940; and the first families moved in February 24, 1940 or 9 months after starting of operation. In this same Pennington Court project, the actual cost of the completed work was about $890,000, which was $154,000 under the original estimated cost set up by the Board and Housing Authorities.

CONCLUSION

Our experience with the rigid frame flat plate design has shown that it is possible to design this type of structure with comparative simplicity. Particularly, where considerable similarity of framing prevails, the small amount of extra time involved in precise calculation of the bending moments is insignificant, and results in greater confidence that the structure is economically and properly proportioned for the stresses it will be called upon to bear. As compared with wall bearing projects of similar scope, the resulting structures have proven to be particularly adaptable to architectural arrangement, free from cracking and defects, cheap in cost, and superior in speed of erection. The success attained in Newark and Atlantic City has led to the adoption of this type of structure for the latest and largest housing development in a nearby city.

PERSONNEL

The Newark Housing is being constructed under the Housing Authority of the City of Newark, Charles P. Gillen, Chairman; Neil J. Convery, Executive Director; and James A. Kilgour, Technical Director. The Architects Board of Design consists of J. F. Cook, E. C. Epple, G. E. Jones, F. H. Koenig, J. Sanford Shanley, and M. A. Wolf. In Atlantic City, the chairman of the Atlantic City Housing Authority is Walter J. Buzby; Robert A. Watson, Executive Director; and B. Dudnick, Technical Director; with Howard A. Stout and J. Vaughan Mathis, Associate Architects. The writer's firm acted as Structural Engineers for all these projects, including field supervision. M. P. van Buren of the firm, was in charge of the structural design work.

The General Contractor for the Atlantic City Project was John McShain, Inc. of Trenton, N. J. For the Newark Projects, to date, Pennington Court, Fatzler Co.; Seth Boyden Court, Pellecchia Constr. Co.; Baxter Terrace, Frank Briscoe Co.; all of Newark, N. J.; Stephen Crane Village, L. Neckermann, Inc. of New York City. Wm. T. Gotelli, Inc. of Newark was subcontractor on the reinforced concrete work for three of these projects.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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Tunnel Lining Practice on the Delaware Aqueduct*

By Charles M. Clark and George Spann†

SYNOPSIS

The Delaware aqueduct is a deep rock pressure tunnel, 85 miles long which will carry water from upstate reservoirs to New York City. Throughout its entire length, the aqueduct will be lined with concrete to provide a waterway of circular cross-section 13½, 15 or 19½ feet in diameter. The construction of the aqueduct is now in progress under twelve contracts held by eleven contractors, eight of whom have completed the excavation and are now engaged on placing the concrete lining. Although great variations exist in the methods and equipment employed under the several contracts, results that are satisfactory both as to progress and quality of finished work are being obtained in each case.

This paper gives a general description of the work, the requirements for the cement and aggregates for the concrete, and describes the methods used in batching, transporting, mixing and placing the concrete for the lining. Reference is also made to the special construction required for portions of the tunnel through badly broken or faulted and decayed rock and to the rapid sustained progress made in placing the lining under several of the contracts.

DESCRIPTION OF AQUEDUCT

The Delaware aqueduct now being constructed by the Board of Water Supply of the City of New York is part of the project for furnishing an additional supply of 540 million gallons daily of water from the Rondout creek and tributaries of the Delaware river. It will be a deep rock tunnel 85 miles long. The aqueduct and the Rondout and Neversink reservoirs with their 6-mile connecting tunnel constitute the first stage of the project and will make available 170 million gallons per day. The East Delaware reservoir with the 26-mile tunnel to Rondout reservoir will constitute the second stage

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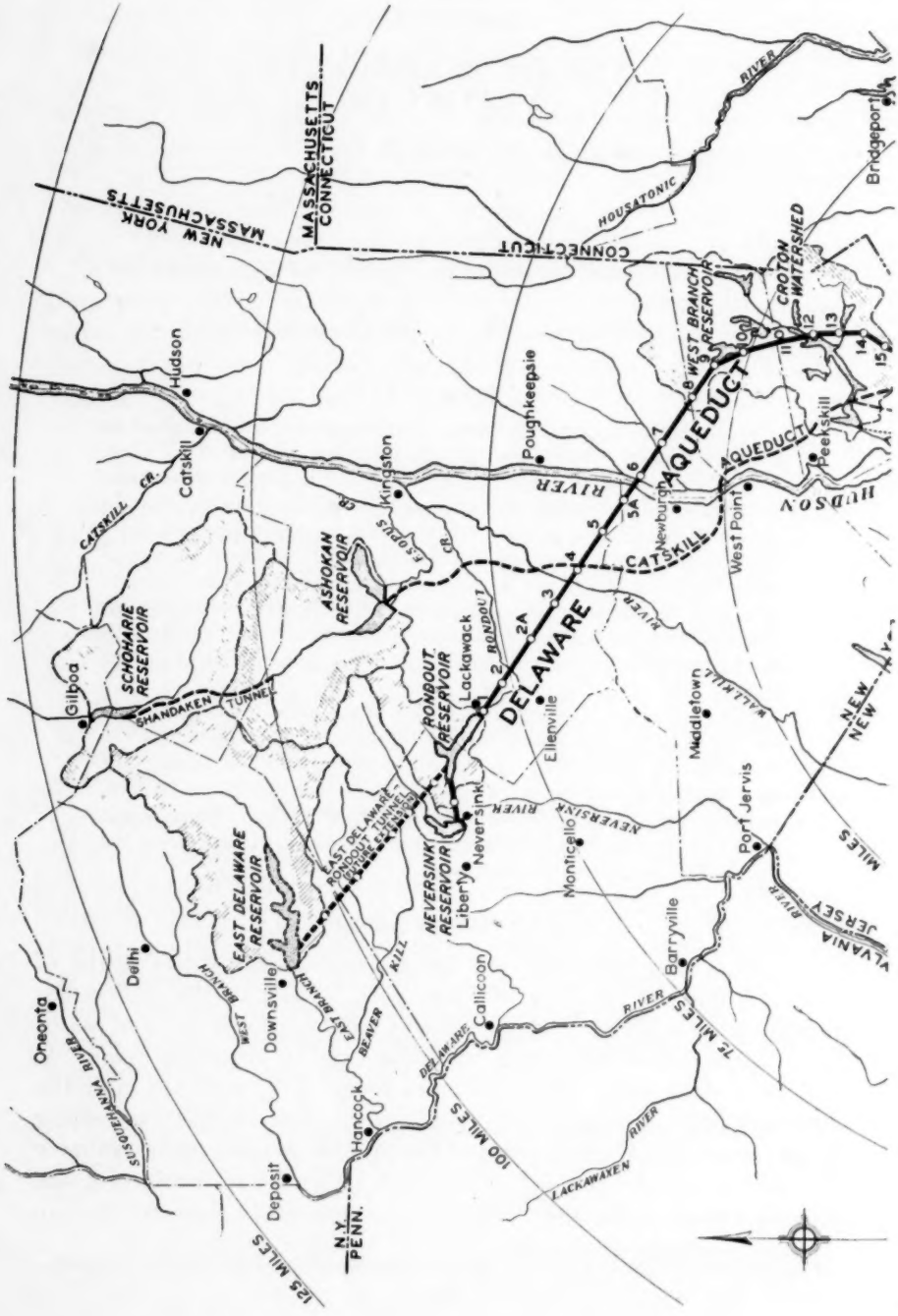
†Board of Water Supply, City of New York—Chief Engineer and Assistant Engineer respectively.


MASSACHUSETTS
CONNECTICUT
NEW YORK
MASSACHUSETTS
CONNECTICUT
HOUSATONIC
RIVER
Bridgeport
CROTON WATERSHED
WEST BRANCH RESERVOIR
AQUEDUCT
AQUEDUCT
HUDSON
RIVER
Hudson
Catskill
CATSKILL CR.
Kingston
ESOPUS CR.
Poughkeepsie
Newburgh
Peekskill
West Point
CATSKILL
ASHOKAN RESERVOIR
SCHOHARIE RESERVOIR
SHANDAKEN TUNNEL
Gilboa
RONDOUT RESERVOIR
Lackawack
RONDOUT
DELAWARE
RIVER
Ellenville
Middletown
WALLKILL
NEW
NEW
MILES
Port Jervis
NEVERSINK RIVER
Monticello
Neversink
NEVERSINK RESERVOIR
Liberty
EAST DELAWARE RONDOUT TUNNEL
EAST DELAWARE RESERVOIR
Delhi
Downsville
WEST BRANCH
EAST BRANCH
BEAVER KILL
Callicoon
Barryville
RIVER
75 MILES
PENNSYLVANIA
NEW JERSEY
RIVER
Oneonta
DELAWARE
Hancock
100 MILES
LACKAWAXEN RIVER
Deposit
N.Y.
PENN.
125 MILES

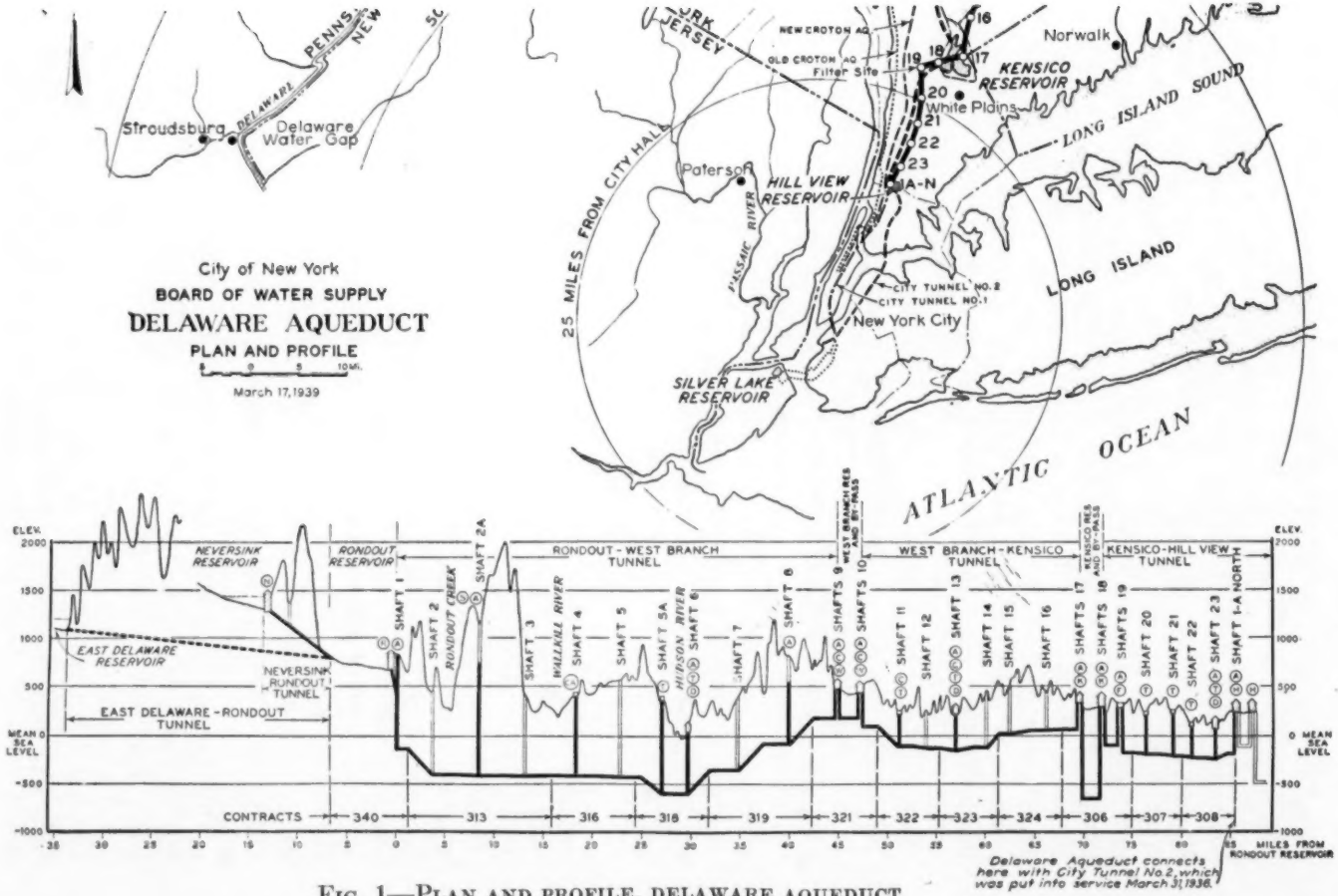


FIG. 1—PLAN AND PROFILE, DELAWARE AQUEDUCT

and will supply an additional 370 million gallons per day. The entire project is mapped in Fig. 1.

The Delaware aqueduct extends approximately 45 miles in a southeasterly direction from Rondout reservoir to the existing West Branch reservoir of the Croton system with a diameter inside the concrete lining of 13½ ft. From West Branch reservoir to Kensico reservoir of the Catskill system, a total distance in a southerly direction including the bypass tunnels under these reservoirs of about 26 miles, the tunnel will have a finished diameter of 15 ft. The 14-mile section from Kensico reservoir to the Hillview distributing reservoir of the Catskill system, just north of the New York City line in Yonkers, will have a finished diameter of 19½ ft.

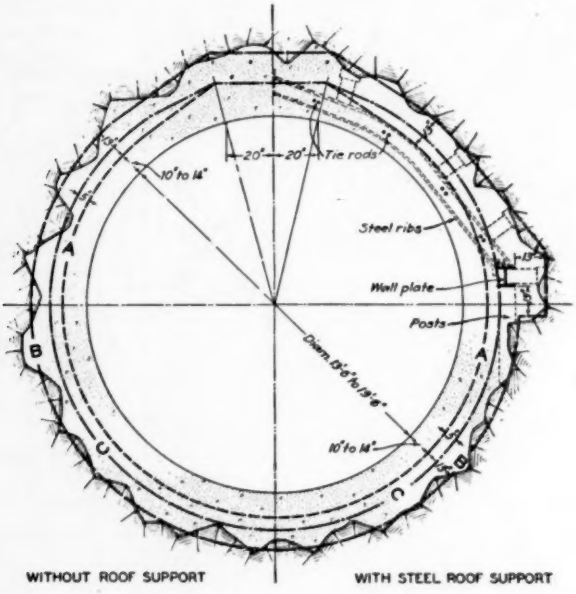


FIG. 2—TYPICAL TUNNEL SECTIONS

THICKNESS OF TUNNEL LINING

The thickness of concrete forming the tunnel lining at any point is dependent on its depth below the ground water level and the character of ground encountered. To insure sufficient minimum thickness of lining, certain lines are prescribed for the tunnel sections. The innermost of these, the "A line" shown on the typical tunnel sections, Fig. 2, is a line within which no rock is permitted to project and is either 10, 12 or 14 inches outside of the waterway depending

upon the external pressure which the lining was designed to withstand. The second line, the "C line," generally 5 in. outside of the first, represents the average effective thickness assumed in the design. Sound rock, steel or other imperishable material is allowed to remain within the "C line" only to a limited extent. Excavation and concrete are paid for to the "B line," generally 13 inches outside the "A line," except that under certain conditions payment at a lower price is made for concrete which is unavoidably in excess of the volume within the "B line."

CONSTRUCTION OF THE DELAWARE AQUEDUCT

The first major construction contracts were awarded early in 1937 and the work has since been vigorously prosecuted through 32 shafts at 26 locations. At the end of December, 1940, 80 miles of tunnel had been excavated and 35 miles of concrete invert and 21 miles of sidewall and arch had been placed. Not all of the shafts are utilized directly in connection with the placing of the tunnel lining, as portions of the tunnel adjacent to one shaft are in many cases lined with concrete delivered through the next shaft to the north or south, thereby avoiding duplication of equipment and freeing the unused shafts for other work. The shafts through which concrete can be brought in are from two to five miles apart.

PORTLAND CEMENT

Portland cement is furnished in accordance with specifications prepared by the late Thaddeus Merriman, former Chief Engineer, subsequently Consulting Engineer, of the Board of Water Supply, as a result of many years of investigations under his leadership and guidance. These specifications* were written shortly before the award of the first construction contracts on the Delaware project and contain provisions not theretofore present in contracts of the Board of Water Supply. Of particular importance are those whose purpose it is to limit the alkali content, to assure thorough burning of the clinker and to prevent partial hydration of the clinker or cement prior to the latter's use in concrete, grout or mortar. Specific provisions included to attain these results are those specifying the approximate temperature at which the kilns shall operate, requiring the use of pyrometers continually recording that temperature, requiring the recording and control of the speed of kiln-rotation, prohibiting the use of water for cooling the cement clinker except while hotter than dull red, limiting the period of storage of the clinker prior to grinding, requiring that the clinker be protected from the weather and kept

*Abstracted in Appendix.

dry, and prohibiting the use of water during the grinding of the clinker.

In the course of Mr. Merriman's investigations, he devised the so-called sugar test which, by measuring the solubility of the cement in a sugar solution, can detect cement that has been either underburned or exposed to moisture. This test is now incorporated in all Board of Water Supply specifications for portland cement as well as a test to indicate resistance to sulphate waters. Proof that cement under these specifications is of higher quality than that obtained in previous work of the Board of Water Supply rests in the superior quality of the concrete, mortar and grout made with the cement. Concrete removed from the work and closely examined has a dense, rock-like structure, lacking the powdery crystalline material which is always present in deteriorated concrete. Laitance, which formerly was always present in varying amounts to the detriment of the finished work, has been practically eliminated.

A cement ratio of two barrels of cement per cubic yard of finished concrete is used for the tunnel lining. This relatively high cement content was adopted to insure concrete which should be as nearly watertight, dense and insoluble as it is practically possible to attain.

Nearly three million barrels of cement have been made to this specification and 2¼ million barrels shipped by 10 mills in New York and Pennsylvania. Except for grouting and some other secondary uses for which cement is handled in bags, all cement for tunnel lining is shipped in bulk in special steel hopper cars, generally of 400 bbls. capacity. At the point of rail delivery the cars are dumped, under roof, into depressed hoppers from which screw conveyors feed bucket elevators which deliver the cement into elevated steel silos of a capacity of several cars. A 400 bbl. car is unloaded, on different contracts, in about 1¾ to 3 hours. From the rail-side silos closed steel bodied trucks transport the cement to the shaft-head where it is end-dumped, under roof, into hoppers and elevated into the silos of the batching or mixing plants. Rail-side and shaft-head storage capacity totalling from about one day's consumption to several thousand barrels is provided to insure continuous operation.

FINE AND COARSE AGGREGATES

All fine aggregate for concrete is natural sand, obtained either from local deposits or from Long Island, thoroughly washed and graded and subjected to tests for acid solubility, for deterioration in boiling water, for organic impurities and for strength in mortar. Coarse aggregate is usually either gravel or broken stone, but a mix-

ture of the two has been used where there was a deficiency in the larger size of gravel. Gravel is obtained from local deposits or from Long Island. Broken stone is either trap rock from New Haven, Conn., or from Haverstraw, N. Y., or a bastard granite from Cold Spring, N. Y., or is selected rock from the tunnel excavation, gneiss, gneissoid granite, or the harder parts of the sandstone, crushed, screened and washed near the shaft-head. No limestone, shale, Manhattan schist or other weak, micaceous or possibly soluble rock is accepted. Coarse aggregate is handled in two sizes 1½ in. to ¾ in. and ¾ in. to ¼ in., which are kept separate until combined in the batching process. All coarse aggregate is washed, graded, examined for the presence of carbonates and shale and subjected to test for solubility in hydrochloric acid. To reduce the number of thin, elongated pieces in the coarse aggregate, approval was given only to those plants furnishing gravel or broken stone that were equipped with secondary crushers of the gyratory type. Table 1 shows the limits of screen analysis which are enforced.

TABLE 1—LIMITS OF SCREEN ANALYSIS

Coarse Aggregate (Gravel or Broken Stone)			Sand	
	Size of Screen, In.	Cumulative Percentage Passing	Size of Screen	Cumulative Percentage Passing
1½ to ¾ in.	1½	95 to 100	¼ in.	100
	1	30 to 70	No. 4	95 to 100
	¾	0 to 8	No. 8	80 to 95
			No. 16	40 to 70
¾ to ¼ in.	¾	95 to 100	No. 30	20 to 50
	½	40 to 80	No. 60	12 to 30
	⅜	10 to 30	No. 100	0 to 3
	¼	0 to 7		

On the three most southerly contracts which are in, or near, high-class residential areas, the crushing of rock at the shaft sites was prohibited. This condition and the ready availability with water transportation of excellent sand and gravel from Long Island and of crushed trap and other rock from nearby points, led to the use of purchased aggregate. For these contracts ample stock piles have been mantained at the points of unloading from barges and at the shaft sites to insure against interruption of production by cold weather and of water and truck transportation. For those up-state contracts where crushing at the shaft site is permitted, large stocks, up to 150,000 cu. yds., have been built up either during tunnel excavation or during the early stages of concreting. Gravel and sand have been stocked either at the points of production, at the shaft sites, or at

both, sufficient to tide over interruptions of truck transportation and winter shut-downs of the washing plants.

BATCHING AND MIXING OF CEMENT AND AGGREGATES

Three general methods for batching, transporting and mixing of concrete are in use.

(1) From a batching plant near the shaft the batch is transported in transit-mix trucks to the shaft-head where water is added and the batch mixed just before it is dropped through a pipe down the shaft to a hopper in the tunnel.

(2) From a mixing plant at the shaft-head the concrete is dropped through a pipe to the hopper in the tunnel.

In both of these cases concrete is commonly transported to the place of use in agitator cars. The agitator car bodies consist of an electric-motor-driven revolving drum. When the car reaches the point of discharge, the motors are plugged in on the power line and the concrete remixed just before it is discharged either directly into the invert or to the concrete pumps, guns or belt conveyor.

(3) From a batching plant at the shaft-head, cement and aggregate are dropped through separate pipes to separate hoppers in the tunnel, whence they are transported in two-compartment batch boxes or cars, with the cement covered and kept separate from the damp aggregate, to a mixer in the tunnel at the point of placing.

Automatic weighing batchers are used for all ingredients. To minimize the escape of cement dust with its nuisance and health hazard, special chutes and hoods are used between the hopper at the bottom of the cement drop pipe and the cement compartment of the car and a 6-in. vent pipe is carried from the hopper to the shaft-head to relieve air pressure in the hopper when a batch of cement is dropped. Mixers in the tunnel discharge directly to the concrete pump or gun or onto belt conveyors which take the concrete to the pump or gun or to the point of deposit in the case of invert concrete. Concrete test cylinders, 6x12-in., representative of the concrete placed in the work, are made at frequent intervals and broken at 28 days.

Relative proportions of sand and of each size of aggregate are varied as necessary to give a workable mix with the materials available on each contract. A fairly typical mix, by weight, is 1 cement, 1.40 sand, 1.42 ¾-in. stone and 1.56 1½-in. stone. The quantity of mixing water is controlled, by telephone, by the inspector at the forms. In general, the proportions are the same for curb walls, invert, sidewall and arch. In some cases the same mix has been used

in reinforced lining; in other cases a mix without 1½-in. stone has been used.

METHODS USED IN PLACING LINING

While there are many variations in detail in the methods of placing the tunnel lining, a typical procedure where concrete is mixed above ground is shown in Fig. 3. Here the lining is placed in four operations; first, concrete curb walls are built on both sides of the tunnel; second, all rails, ties and loose rock are removed from the bottom of the tunnel between the curb walls; third, the invert concrete is placed; and finally, the side walls and arch are poured in one operation which completes the lining. The usual practice is to complete each of these operations in turn for a major portion of the tunnel, such as from the contract limit to a shaft or from one shaft to the next, before beginning the next operation. Two important variations of this general procedure have occurred. In one case, all operations are advancing simultaneously, in successive lengths of 150 ft. or more, in the same section of the tunnel; in the other, the side walls and arch are poured next after the curb walls and before the tunnel bottom is cleaned and the invert poured.

REMOVAL OF LAGGING AND BLOCKING AND CLEANING OF ROCK

Immediately before concreting, all lagging and blocking and other timber not essential to the safety of the roof is removed. Steel ribs, steel wall plates and timber posts supporting wall plates are left in. Any lagging which it may be necessary to leave in is of steel.

All rock against which concrete is to be placed is thoroughly cleaned. Broken rock in the bottom, which cannot be reached by the excavating machinery, is removed by pick and shovel or blasted out with jets of high pressure air. Final cleaning is by hose streams, high pressure air or high-pressure air-and-water blasts. Clean-up water is removed from depressions in the bottom by small portable pumps just before concreting.

CURB WALLS

Of the eight contractors now engaged in tunnel lining six, as a construction detail, make use of curb walls. These are placed behind wooden forms set accurately to line and grade. They furnish support for the invert or arch forms and provide foundations for rails carrying equipment for the following concreting operations. Concrete for the curb walls is mixed above ground, transported in either agitator cars or side dump cars and chuted into place. Where curb walls

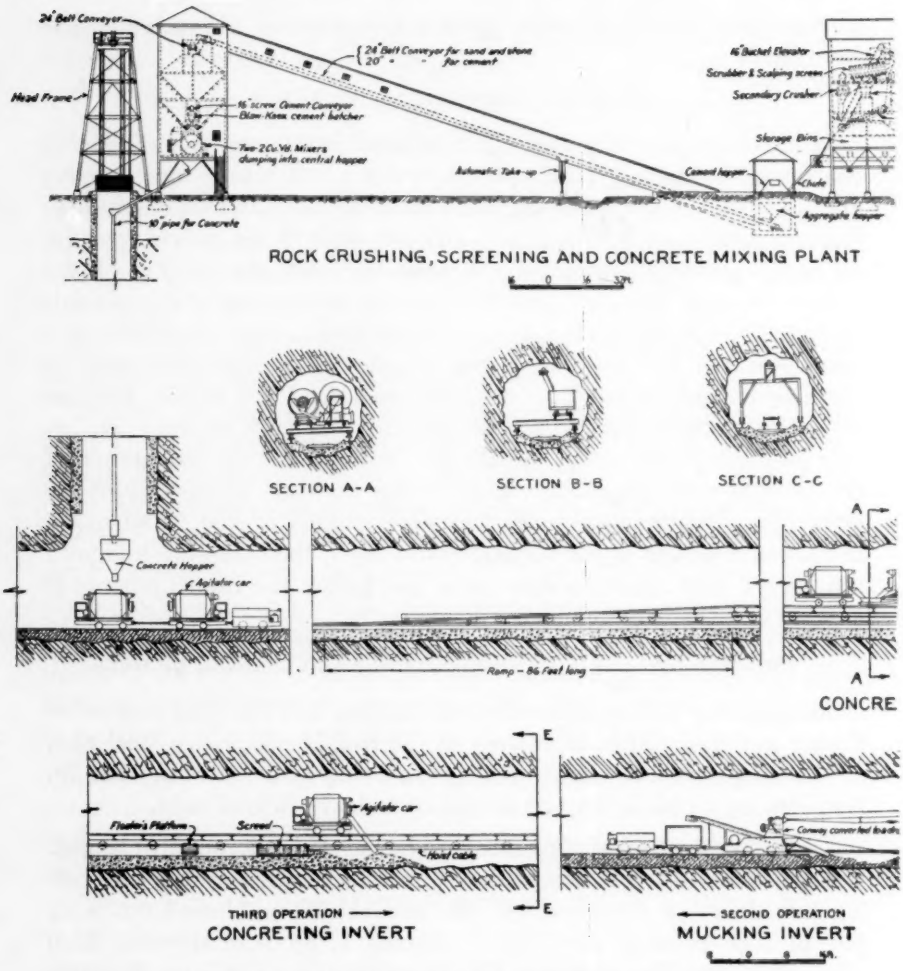


FIG. 3—TYPICAL CONSTRUCTION PROCEDURE

(The drawing is continuous across opposite page)

have not been constructed, rails for carrying equipment are supported on piers constructed of bagged concrete.

PLACING OF INVERT CONCRETE

In some cases the track is relaid following the cleaning of the bottom, the ties being supported on bagged concrete piers, thus permitting the concreting of the invert to start at a point distant from the source of concrete supply. In other instances no track is relaid upon the cleaned rock invert, the placing of the concrete invert

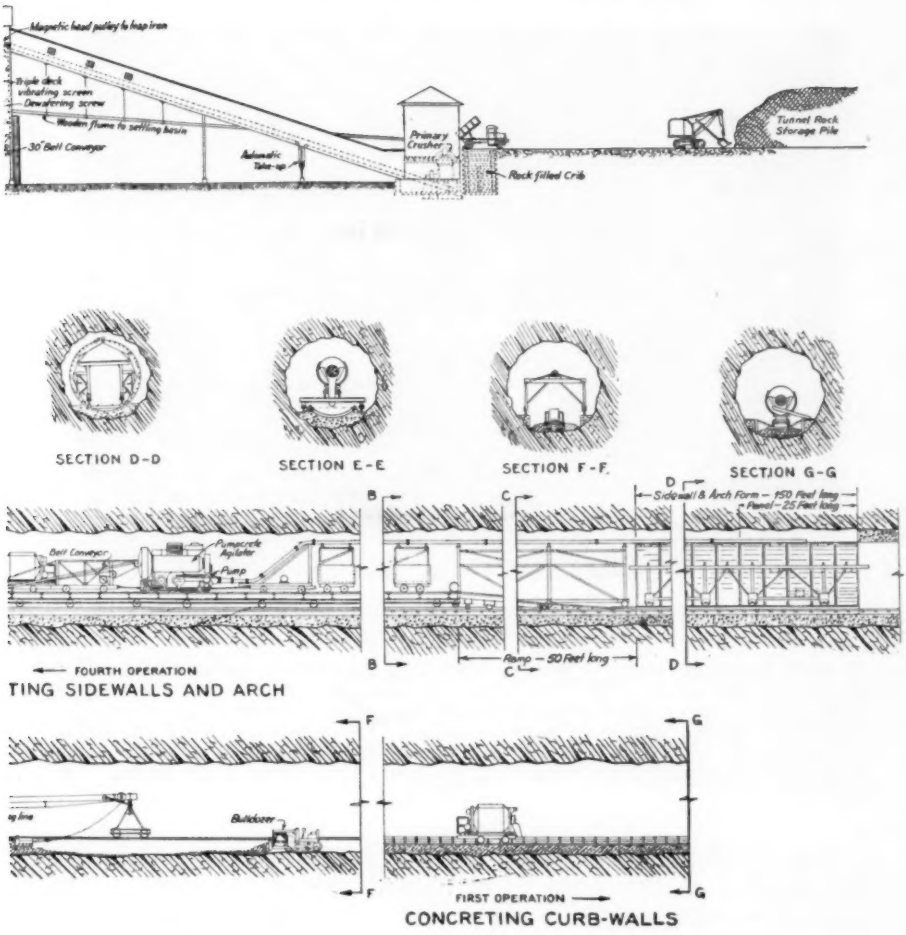


FIG. 3—TYPICAL CONSTRUCTION PROCEDURE

(The drawing is continuous across opposite page)

starting near the shaft and proceeding away from the shaft, track being relaid on the finished concrete invert.

Several methods of placing the concrete in the invert of the tunnel are employed. Where previously mixed concrete is brought into the tunnel, agitator cars are run up a ramp to the invert bridge which is a steel framework mounted on flanged wheels riding on rails on the curb walls. From track on the invert bridge the agitator cars discharge the concrete through chutes directly into the invert just ahead of the invert screed as shown on Fig. 4 and 5.

FIG. 4—DELAWARE AQUEDUCT. PLACING CONCRETE INVERT IN 15-FT. TUNNEL NORTH OF SHAFTS 17

In north tunnel at Shafts 17 looking down from the movable bridge at the newly-placed concrete invert. The men below the bridge are on a steel screed which is being pulled forward, leaving the completely-formed invert seen in the foreground. The final troweled finish is given to the surface by masons on a movable platform, not seen in the photograph, that follows some distance behind the screed. A five-cubic yard Jaeger agitator car has just delivered its load ahead of the screed. Jan. 25, 1940.

Steel forms supported on the curb walls and set accurately to line and grade form the radial joints at each side of the invert. The finished concave surface of the invert is formed by a steel screed riding on and guided by the steel side forms and pulled ahead by a cable operated by a tugger hoist. Men working on and in front of the screed aid in distributing the concrete and men riding on the tail of the screed fill any voids left by the passage of the screed, give a preliminary finish and set inserts for the holding-down bolts for the sidewall forms.

The finishers work from a following platform which also travels on the side forms. Where mixed in the tunnel, the concrete is delivered from the mixer either by a pump-crete machine and delivery pipe to the point of deposit, or by a belt conveyor discharging directly over the invert at the point of deposit. Where the belt conveyor is used, mixer and belt conveyor are moved as the work advances, the direction of progress being toward the shaft through which the cement

FIG. 5—PLACING CONCRETE INVERT IN 15-FT. TUNNEL SOUTH OF SHAFT 14

South tunnel at Shaft 14 looking north. Invert concrete is being placed on the clean rock bottom between curb walls. The screed in the background is moved ahead by means of the wire rope shown in the picture. The steel beams in the upper portion of the photograph are parts of the invert bridge. Agitator cars, traveling on rails supported by the invert bridge, deliver the mixed concrete directly to the invert. The forms for the radial joints between the invert and the side walls are on top of the curb walls. Nov. 12, 1940.

and aggregates are delivered to the tunnel. Under this method, no invert bridge is required. Where concrete is pumped to the point of deposit, the delivery pipe is laid on an invert bridge and the addition of sections to the pipe makes it possible to leave the mixer, pump-crete machine and appurtenant equipment in one position until a section, the length of the invert bridge, has been poured. Except on one contract, the invert in each case is formed by the use of a steel screed as above described. In the exceptional case, the invert concrete is placed under forms which are braced to the rock above, the concrete being delivered under the forms by pump-crete machine and pipe.

PLACING SIDE WALL AND ARCH CONCRETE

All concrete for the lining above the invert is placed either by concrete guns or by concrete pumps, Fig. 6, and the entire section above the invert is placed in one operation. The side wall and arch con-

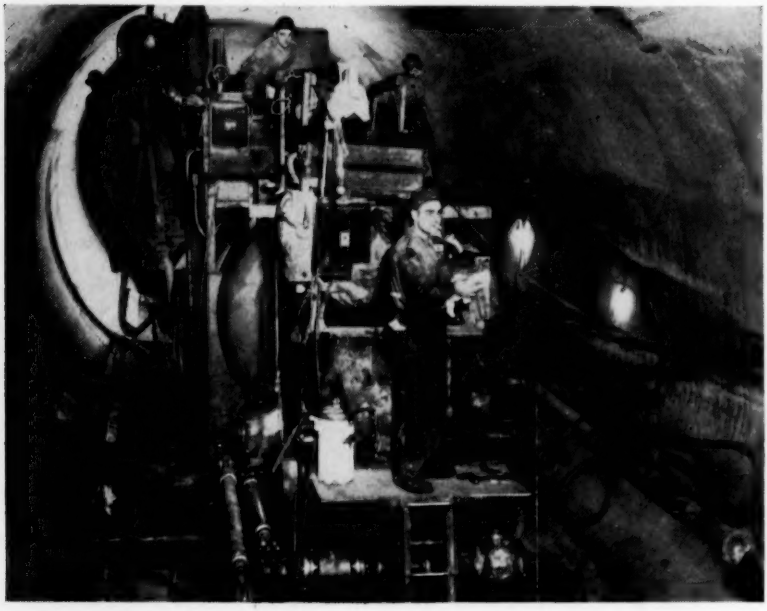


FIG. 6—PUMP-CRETE MACHINE IN 19½-FT. TUNNEL SOUTH OF SHAFT 20

Pumpcrete machine in south tunnel at Shaft 20. The cement and aggregates are delivered to the mixer at the far end of the machine. The mixed concrete passes through the agitator shown in the center of the picture and enters the pump cylinders from which it is forced into the delivery pipe through the two outlets at the bottom of the photograph. The completed lining of the tunnel shown in the picture has an inside diameter of 19½ ft. Oct. 31, 1940.

crete is placed either as a continuous pour covering a week of five working days or in definite lengths of one day's work. In the continuous pour method, sufficient telescoping forms are provided so that those used at the beginning of the pour can be stripped, moved ahead and re-used without interrupting the continuity of the pour. As the contracts prohibit work on Sundays and holidays except in emergencies, it is necessary to discontinue operations on these days. Telescoping steel forms, hinged so as to be readily collapsed to a size which will permit them to be moved forward through other forms in place, are used for continuous pours and in some other cases. Fig. 7. On some contracts, where the side-wall and arch forms are moved ahead in one operation once a day, non-telescoping forms are used, the form section at the advance end of the work being moved first. See Fig. 8. Some type of carriage running on broad gauge track laid on the finished invert astride the concrete track is used for handling the

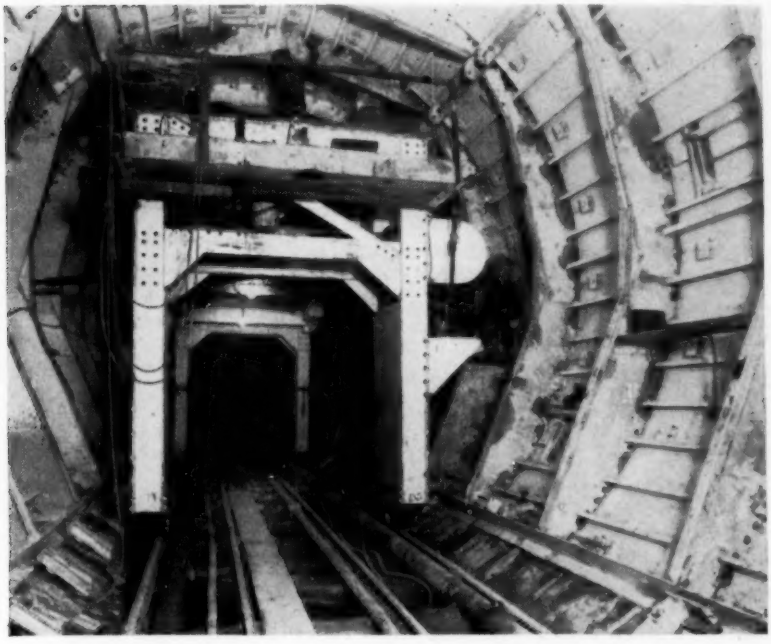


FIG 7—FORMS AND FORM CARRIAGE FOR SIDE WALLS AND ARCH OF LINING OF 19½-FT. TUNNEL NORTH OF SHAFTS 19

In north tunnel at Shafts 19 looking toward Shafts 18. The form carriage has just placed in position a 20-ft. length of form for the 19½-ft. diameter arch and side wall concrete lining. The section of form when collapsed rests on hydraulic jacks at the top of the carriage and the sides are drawn in by hand-operated ratchet screw jacks, one of which may be seen at the left of the carriage. To place in position the hydraulic jacks raise the forms and the hand-operated screw jacks push them out. At the bottom, the forms are pulled tight against the invert concrete by bolts which are screwed into inserts previously placed in the invert concrete. Four of these bolts with steel clamps may be seen on the left. July 9, 1940.

forms in units 20 to 30 ft. long. Hydraulic and screw jacks on the carriage collapse the forms for moving and expand them to the position for the pour. The forms are held in place by bolting the bottom edge of each side to inserts built into the invert. To minimize the cutting of openings for grout and drain pipes, holes closed by plugs flush with the face of the form are provided in standard locations and removable plates or doors provide access behind the form for vibrating the concrete, for inspection and for other purposes.

All concrete for sidewall and arch is delivered, through one or two pipes, at or near the top of the form and allowed to take its natural slope, about 1 on 4, along the form. Generally, the end of the concrete pipe is kept deeply buried to force the concrete up through

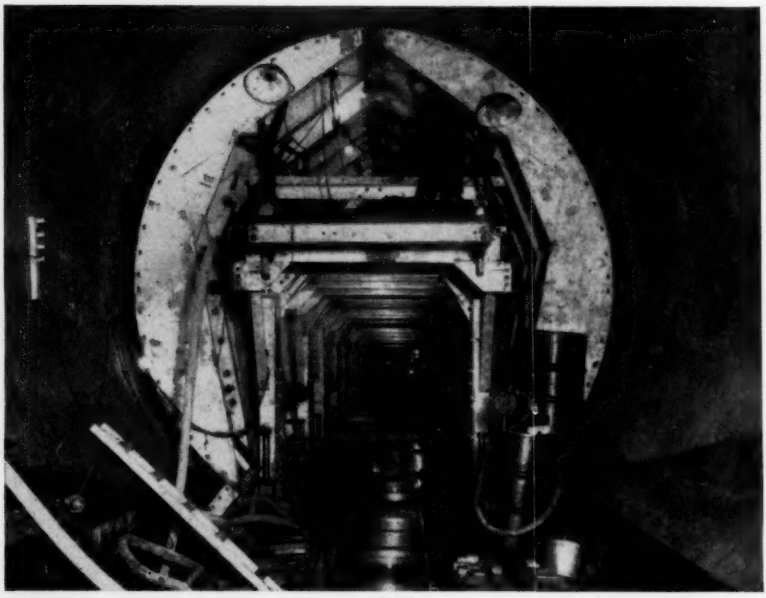


FIG. 8—FORMS AND FORM CARRIAGE FOR SIDE WALLS AND ARCH OF LINING OF 15-FT. TUNNEL SOUTH OF SHAFTS 10

View in wouth tunnel at Shafts 10 looking north. A section of forms for the 15-ft. diameter arch and side walls has been loosened and partly pulled away from the lining. Jacks for setting and loosening the forms are plainly visible. The total length of the forms and also of the form carriage is about 150 ft. and the length of a single pour for the side walls and arch is limited to the length of the forms. After the concrete has set sufficiently, the forms are loosened and moved ahead in sections. The side walls are constructed on the previously placed curb walls prior to the placing of the invert concrete. Nov. 7, 1940.

the steel roof support and into overbreaks. With the pump, air is generally used only when filling the last 5 ft. or so of arch at a bulkhead. On some contracts, the welded steel concrete pipe and the carriage mounting pump or gun are backed out as a unit as the concrete over the form advances; on others, the pipe is shortened a section at a time. One electric internal vibrator is used on each side of the form, men working behind the forms or through doors, as most convenient. A limited amount of spading is also done. The forms are not vibrated. Stripping time varies from a maximum of about 10 to 12 hours for both continuous pours and short daily pours to a bulkhead, to a minimum of 6 hours.

REINFORCEMENT AND STEEL INTERLINING

In several places, in badly broken or faulted and decayed rock, the tunnel is being built with a water-tight steel plate shell lined

and surrounded with concrete, or with a reinforced concrete lining, or with both the steel-plate interlining and a reinforced outer lining. The total length of interlining and the total length of reinforced lining each amounts to less than one per cent of the length of the tunnel. The usual procedure has been first to erect the reinforcement, if any, then to pour the sub-invert of the outer lining, 1 to 2 inches outside the position of the steel plate interlining, with two embedded steel skid rails projecting above the concrete to the position of the interlining, then to assemble the interlining in 4-foot rings in an erection chamber, an enlargement of the tunnel at one end of the special section, then to skid the rings into place on the embedded rails, rivet the rings together and seal the joints by welding, then to concrete outside the interlining and grout between the interlining and the subinvert and finally, to pour the inner lining. The concrete in the reinforced section is placed either by longitudinal pipe between the top of the interlining and the reinforcement in the usual way, or in one case it is blown through 6-in. standpipes extending up into the overbreak above the reinforcement, the pipes being welded to the steel interlining at the crown and terminating at their upper ends in 6-in. elbows pointing back toward the end bulkhead or previously placed concrete. In one case the outer reinforced lining was poured with wooden forms to give a 4-in. clearance within which the steel-plate interlining was skidded; the outer concrete was blown through a pipe between form and reinforcement in the usual way; the space around the steel interlining was grouted; and the inner lining placed in the usual way.

CONTROL OF INWARD LEAKAGE

Inflows of water encountered during excavation were in many cases stopped by grouting at the time the tunnel was driven. Where this was not the case, in order to protect the fresh concrete, sheet steel "pans" were fastened and calked against the seeping rock and the water led through the forms in steel pipes. These pipes were later used for grouting the spaces behind the sheet steel pans and the crevices in the surrounding rock.

PROGRESS IN CONSTRUCTION OF TUNNEL LINING

Excellent progress has been made in placing the concrete lining of the tunnels. Table 2 gives the lengths of tunnel included under each contract and the amounts of invert and combined side walls and arch completed to Dec. 25, 1940.

Although record pours of concrete in one day or during one week may be more spectacular, outstanding progress figures such as given

TABLE 2—PROGRESS OF TUNNEL CONTRACTS

Contract	Contractor	Total Length of Tunnel	Diameter of Tunnel	Length of Tunnel Excavated to Dec. 25, 1940		Concrete Lining Placed to Dec. 25, 1940			
						Invert		Sidewalls & Arch	
		Feet	Feet	Feet	Per Cent	Feet	Per Cent	Feet	Per Cent
340	Mason & Hanger Co., Inc.	8,307	13.5	552	7	0	0	0	0
313	Samuel R. Rosoff, Ltd.	75,157	13.5	66,768	89	0	0	0	0
316	Walsh Construction Co.	48,145	13.5	48,145	100	36,521	76	0	0
318	Pleasantville Constructors, Inc.	36,678	13.5	36,678	100	9,206	25	0	0
319	Dravo Corporation	55,870	13.5	41,941	75	0	0	0	0
321	Frazier-Davis Construction Co.	34,962	13.5 & 15.0	34,962	100	580	2	5,562	16
322	The Utah Construction Co.	33,400	15.0	33,400	100	23,519	70	8,867	27
323	Seaboard Construction Corp.	30,700	15.0	30,700	100	30,284	99	12,757	42
324	S. A. Healy Co.	37,100	15.0	36,664	99	0	0	0	0
306	Associated Contractors, Inc.	34,758	15.0 & 19.5	34,758	100	34,319	99	30,753	88
307	S. A. Healy Co.	27,000	19.5	27,000	100	22,993	85	22,569	84
308	J. F. Shea Co., Inc., and Henry J. Kaiser Co.	29,094	19.5	29,094	100	28,271	97	28,005	96
	TOTALS	451,171		420,662	93	185,693	41	108,513	24

below covering long stretches of tunnel appear more useful. In placing the concrete invert of the 13½-foot tunnel between Shafts 5 and 4 under Contract 316, an average progress of slightly more than 2500 ft. per week was maintained during five consecutive weeks. In the 15-ft. by-pass tunnel between Shaft 17 Downtake and Shaft 18 Uptake the concrete invert was placed for four consecutive weeks at an average rate of slightly less than 2800 ft. per week under Contract 306. Also under Contract 306, the combined sidewalls and arch between Shafts 19 Uptake and 18 Downtake, 19½ ft. in diameter, were placed at an average rate of approximately 1660 ft. per week for five consecutive weeks. Under Contract 307, between Shafts 21 and 20, where the invert and arch are advancing together, progress has been at a steady rate averaging over a period of fifteen consecutive weeks about 700 ft. per week of complete tunnel lining.

APPENDIX

Abstract of Specification for Portland Cement of the Board of Water Supply of the City of New York, Feb. 1, 1940

Portland Cement—

Sect. 15.1—Under Item 15 the contractor shall furnish, deliver and place in the work portland cement that has been manufactured at established American plants of such recognized capacity and quality of output as to insure a product at least equal to that hereinafter specified. Unless otherwise permitted, cement from not more than three plants shall be used, and, in general, only the product of one plant shall be used at the same time in any section of the work. Within 15 days following the notice to begin work the Contractor shall submit for the approval of the Engineer the names of the plant or plants from which he proposes to furnish the cement and no

cement shall be furnished from any plant until it has been approved. At the same time, he shall also furnish to the Engineer an estimated schedule of the monthly deliveries he will require from each of the said plants during the ensuing four months. This schedule shall be revised by the Contractor every 60 days thereafter.

General Requirements—

Sect. 15.2—The cement and the clinker from which it is made will be subjected to thorough inspection, to tests, and to frequent analyses as provided in the following sections. The cement when delivered and used on the work shall be dry and free from lumps and caking.

Requirements of Manufacture—

Sect. 15.3—The raw materials from which the cement clinker is made shall be thoroughly blended and finely and uniformly ground; either the dry or the wet process may be used. The flue dust from the kilns shall not be returned to the raw mix, but shall be rejected unless the nature of the raw materials and of the flue dust is such that the alkalinity of the finished cement is less than 3.8 and its content of free alkali is less than 3.5. (See Section 15.8 for test procedures.) The kilns shall operate at about 2,700 degrees Fahrenheit and a continuous record of the temperature shall be kept. A regular record shall also be kept of the rates of kiln rotation; these records shall be available to the Engineer and are called for in order to insure the greatest possible uniformity of kiln output. The speed of kiln rotation and the kiln temperature shall be controlled and co-ordinated so as to produce a hard and completely burned clinker in which the combinations between the several oxides it contains have been substantially completed. Only a minimum of water may be used as an aid for cooling the clinker while it is hotter than dull red; at all other stages it shall be kept dry and protected from the weather. In general, the clinker shall be ground immediately after it has been made; occasionally, it may be stored, but not for a period longer than six weeks. Grinding aids shall not be employed unless they have been declared to and approved by the Engineer. No water or steam shall be permitted to come into contact with the cement or applied to it at any stage of its manufacture. The gypsum or other source of SO_3 shall be reasonably dry when mixed with the clinker before grinding. No material or admixture shall be added to the raw material, to the clinker, or to the cement at any stage of the manufacturing process without the approval of the Engineer, except that gypsum, anhydrite or plaster may be added in quantities sufficient to secure the necessary percentage of sulphuric anhydride. Water may be added to the clinker only as above provided.

Fineness of Grinding; Soundness of Neat Cement Pats; Time of Set—

Sect. 15.4—The cement shall be ground so that at least 90 per cent, but not more than 98 per cent, by weight, will pass the standard No. 200 sieve. These limits shall not be exceeded. Pats of neat cement, in the steam test for soundness, shall remain firm and hard and shall not distort, check, crack or disintegrate. Similar pats kept in the moist closet for 7 days shall remain firm and hard and unchanged. The cement shall not develop its initial set under the Vicat needle in less than 45 minutes or in less than 60 minutes when the Gilmore needle is used. The final set shall be attained in not more than 8 hours.

Tensile Strength—

Sect. 15.5—The average tensile strength of not less than three standard briquettes made of neat cement and mixed to normal consistency shall not be less than the limits stated in the following table:

Age at Test in Days	Storage of Briquettes	Tensile Strength, Pounds per Square Inch
1	1 day in moist closet	300
7	1 day in moist closet, 6 days in water	600

The average tensile strength of not less than three standard mortar briquettes made of one part of cement and three parts of standard Ottawa sand, measured by weight, shall not be less than the limits stated below:

Age at Test in Days	Storage of Briquettes	Tensile Strength, Pounds per Square Inch
7	1 day in moist closet, 6 days in water	275
28	1 day in moist closet, 27 days in water	375

The average tensile strength of the mortar briquettes at 28 days shall be at least 15 per cent greater than the average tensile strength at 7 days.

Composition of Cement—

Sect. 15.6—The composition of the cement shall be such that the chemical analyses will show it to meet the limits stated below:

The percentage of silica (SiO_2) shall not be less than 21.50;

The percentage of alumina (Al_2O_3) shall not be greater than 5.60;

The percentage of magnesia (MgO) shall not be greater than 4.00;

The percentage of sulphuric anhydride (SO_3) shall not be greater than 1.75;

The percentage of loss on ignition shall not be greater than 0.90;

The percentage of insoluble residue shall not be greater than 0.30;

The ratio between the percentage of alumina (Al_2O_3) and the percentage of ferric oxide (Fe_2O_3) shall not be greater than 1.60 nor less than 1.20;

The ratio between the eprcentage of lime (CaO) and the percentage of silica (SiO_2) shall not be greater than 2.90;

The molecular ratio (Colony's ratio) shall not be greater than 2.60 unless approved by the Engineer. This ratio is obtained by dividing the molecular ratio of the calcium oxide by the sum of the molecular ratios of the silica, the alumina and the ferric oxide.

Chemical Analyses and Physical Tests—

Sect. 15.7—All chemical analyses and physical tests specified in the preceding sections shall conform to the following procedures: Chemical analyses shall be made by the methods prescribed in the United States Bureau of Reclamation Specification 566 for cement for the Boulder Dam, while the physical tests shall follow the methods and procedures of Sections 13 to 45 of Specifications C77-32 of the American Society for Testing Materials, entitled "Standard Methods of Sampling and Testing Portland Cement." All limits prescribed in these specifications include all so-called "tolerances." At least 14 days shall be allowed for the completion of the 7-day tests, and the 28-day tests shall be completed within 32 days.

Alkalinity—

Sect. 15.8—The alkalinity of the cement shall not be greater than 3.8 and its content of free alkali shall not exceed 3.5. These characteristics shall be determined as follows: Weigh out 800 grams of cement and put into an enameled saucepan with

500 c.c. of distilled water. Stir frequently for two hours, then filter through large folded filter paper for 10 minutes. If filtrate is not clear, refilter. Titrate 25 c.c. of the filtrate with N/2 HCl, using methyl orange as the indicator. The number of c.c. of acid required to neutralize the filtrate to the methyl orange end point is the measure of the alkalinity.

Free Alkali Content—

The free alkali content of the cement shall be determined as follows: Measure out 100 c.c. of the filtrate obtained in the alkalinity test of the preceding paragraph into a small beaker, and add 30 to 35 c.c. of a saturated filtered solution of $Ba(OH)_2$. Let stand, filter and wash with H_2O. Pass CO_2 into the filtrate for five minutes. Let stand, filter and wash the precipitate with H_2O. Heat to boiling and, if a precipitate forms, filter it out. Then boil the total filtrate plus wash water down to about 50 c.c., filter it and make it up to 100 c.c. with distilled water. Now take 25 c.c. of the solution and titrate it with N/2 HCl in the presence of methyl orange. The number of c.c. of the acid required to neutralize the 25 c.c. to the end point of the methyl orange is the measure of the free alkali content. Both of these tests are to be made at room temperature.

Sodium Sulphate Test—

Sect. 15.9—The cement will be subjected to the sodium sulphate test. In this test the slabs shall, at the end of 28 days, remain firm, hard and strong and shall show no signs of disintegration or softening. Slight warping or small surface cracks on the edges and corners will not be ground for rejection.

Test Procedure—

The test procedure is as follows: One hundred and fifty grams of the cement are mixed with 43 per cent by weight of water and stirred for one minute by a rotary stirrer in an earthenware jar about 3 inches in diameter and 4 inches high. Near the bottom of this jar is a $\frac{3}{8}$-inch diameter hole closed with a cork. When the stirring is complete the mixture is rapidly poured, by withdrawing the cork, onto a sheet of moistened paper as made by the Paterson Parchment Paper Company, of Bristol, Pa., 30-pound 400 Parchment, white, 24 inches by 36 inches, 34.0 pounds per 500 sheets, or equal, cut to 8 inches by 10 inches, the paper being supported on a clean glass plate 8 inches by 10 inches, in size. On this paper at convenient positions are placed six $\frac{3}{16}$-inch steel balls held in position by paraffin shavings. The poured slab assumes an oval shape about 4 inches by 8 inches in size. A second glass plate also having a moistened parchment paper on its face, is then gently lowered into position on the steel balls so as to exclude all air bubbles. A light weight (two soundness pats are sufficient) is then placed on top of the upper glass plate and the assembly carefully put into the moist closet. Twenty-four hours thereafter, the assembly is put into the water storage tank (circulating water). After another twenty-four hours the glass plates and papers are removed and the slab carefully sawn to size, 2 inches by 4 inches, with a hack-saw and templet, it being kept under water except during the sawing process. At this time a small hole is also drilled through the slab at $\frac{1}{2}$ inch from one end and on its center line. This hole serves to suspend the slab in the test solution. The slab then goes back into the water storage tank for 24 hours, when it is removed and hung on a copper hanger in a 2-quart Mason jar containing 1,500 c.c. of a 10 per cent, by weight, solution of anhydrous sodium sulphate (Na_2SO_4). The slab is suspended so that its top is one inch below the surface of the solution. About ten drops of an alcohol solution of phenolphthalein are added to

the sulphate solution and, after the slab has been placed therein, it is titrated every 24 hours to the phenolphthalein end point with 6N sulphuric acid. During the test the jars are covered with glass tops without washers, except when the titration is being done. During that process the slab is removed and hung in an empty jar close at hand. A daily record of the quantity of acid added is kept. A sudden increase in the quantity of acid consumed is an indication of failure of the slab before cracks visible to the eye appear. This test is to be made at room temperature.

Sugar Test—

Sect. 15.10—The sugar solubility of the cement shall not be greater than 8.0 to the phenolphthalein end point, nor greater than 10.0 to the final clear point. These values shall be determined as follows: A sample of about 100 grams of the cement is passed through the 200-mesh screen and put into a glass bottle closed with a rubber stopper. From this bottle 15 grams are then weighed out and placed into a Nessler tube containing 100 c.c. of a 15-per cent solution of cane sugar in distilled water (commercial granulated sugar such as "Jack Frost"); this solution shall not be more than three days old. The tube and its contents are then quickly shaken by hand and placed on a wheel revolving about 60 times per minute. At the end of about 1 hour and 50 minutes the mixture is poured into a filter paper and funnel and allowed to filter for ten minutes when the beaker containing the filtrate is removed. (In case the filtration time of 10 minutes is too short to produce a volume of filtrate of 30 c.c., it may be lengthened by shortening the time of shaking, but the total time from the putting of the sample into the solution to the end of the filtration must be exactly 2 hours.) Twenty-five c.c. of the filtrate is now titrated with N/2 HCl in the presence of phenolphthalein and the number of c.c. of acid to the end point is the first measure of the sugar solubility. At this stage of the test, in the case of a thoroughly burned cement which has been kept dry, the solution will be practically clear and only traces of ferric oxide and alumina will be in suspension. In the case of an under-burned cement, or one which has been exposed to moisture, the solution will be heavily clouded and the end point must be approached slowly and with caution. When so performed, the phenolphthalein end point can be definitely determined, as the color changes from light pink to yellow. The titration is then continued until the solution is crystal clear and nothing remains in suspension. The total number of c.c. of acid from the beginning of the titration to this final clear point is the second measure of the sugar solubility. This test shall be made at room temperature. In addition to disclosing the quality of the cement as above stated, this test further indicates the character of the hydration products which will be realized in the completed concrete.

Shipments of Cement—

Sect. 15.11—Cement shall be delivered in strong, well-made 4-ply paper packages, each plainly marked with the manufacturer's brand. The weight of all packages shall be uniform. Packages received in broken or damaged condition,shall be rejected or accepted only as fractional packages. If so required, each package of cement shall be sealed or stamped at the expense of the Contractor, under the supervision of, and with a seal or stamp furnished by, the Engineer. With the approval of the Engineer, cement may be delivered in bulk if the Contractor provides approved storage, weighing devices, and all other necessary facilities to insure keeping the cement in good condition and affording a correct measure of the cement used in each batch, as well as in total quantity. Bulk shipments shall be made only in clean cars

or containers. Any such in which lime has been shipped shall not be used unless especially cleaned.

* * * * * * *

Inspection and Tests—

Sect. 15.13—The Engineer shall be notified by the Contractor when the cement is to be manufactured, and the Engineer shall have the liberty at all times to inspect the materials, the processes of manufacture and the laboratory records of analyses and tests made at the cement works, and to supervise the packing.

Samples—

Samples will be taken at the place of manufacture by the Engineer and sent to the Board's laboratory, where the tests and analyses herein specified will be made. Samples shall be stored and shipped in moisture-proof air-tight containers which will be furnished by the Board. These containers shall be crated, ready for shipment, by the Contractor and shall be delivered by him to the express company. Express charges will be paid by the Board.

Notice of Shipments of Cement—

Sect. 15.14—The Contractor shall notify the Engineer when orders for cement are placed and shipments are to be made in ample time (at least 48 hours) to enable him to have his representative present to observe the burning of the clinker, the manufacture of the cement, its loading for shipment and to obtain the necessary samples. In case additional tests become necessary, the Contractor shall rehandle the cement in the storehouse for the purpose of obtaining samples as directed.

Sampling and Storage—

In general, the cement shall be sampled from the conveyor belt at the mill by a continuous sampler, one sample being taken for every 200 barrels or more as determined by the Engineer. Each sample shall weigh 5 pounds. If sampled from the bin proper, tubes shall be used. If a bin is full, it shall be sampled from the discharge openings. The methods of sampling must at all times be approved by the Engineer. All cement stored at the mill shall be kept in sealed silos or other approved bins.

Retesting; Rejected Cement—

Cement kept in store at the mill for more than six months may be retested. Cement failing to pass such tests shall be rejected. If any cement proves unsatisfactory and portions of it have been used in the masonry, such masonry shall be ordered removed and replaced with masonry built of acceptable cement at the Contractor's expense. Test-cylinders from concrete or mortar being used in the work shall be made by the Contractor at any time for purposes of test, if so directed.

Rejection or Acceptance of Cement by Engineer—

Sect. 15.15—Cement may be rejected at the discretion of the Engineer if it fails to meet any of the requirements of these specifications. Cement may be accepted without awaiting the results of the 28-day tests when, in the judgment of the Engineer, the 7-day tests are met and there is definite justification, based on experience and actual plant performance, that the 28-day tests will be satisfactory.

Storehouses for Cement—

Sect. 15.16—The Contractor shall provide suitable storage for cement at approved places convenient to the work, and the cement shall at all times be carefully and in a workmanlike manner protected by him against moisture and exposure to the air.

Cement storehouses shall be weather-tight, shall have tight floors a proper distance above the ground, shall be large enough to admit of keeping on hand a sufficient supply of cement to prevent delays or interruptions to the work and shall have sufficient floor space for storing each carload or barge load of cement separately and affording convenient access thereto for sampling, counting of packages and removal. Cement in packages shall not be piled to a height exceeding 7 feet.

Scales for Weighing—

Suitable, accurate scales shall be provided by the Contractor for weighing the cement in each storehouse and elsewhere on the work, if required, and he shall also furnish all necessary test weights. Bulk cement shall be stored in weather-proof bins or silos, plans of which shall be submitted by the Contractor for the approval of the Engineer and they shall be efficiently operated and maintained by the Contractor. All such storage bins shall be constructed so that there will be no dead storage. The Contractor shall not use cement directly upon its receipt whenever cement previously received has been in store more than 15 days.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL Supplement concluding this Proceedings volume

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by Clark and Spann:

Tunnel Lining Practice on the Delaware Aqueduct*

BY E. N. VIDAL, WALTER H. WHEELER, O. W. IRWIN, J. C. PEARSON, LEWIS H. TUTHILL, ARTHUR A. LEVISON AND GEORGE SPANN

CONVENTION DISCUSSION

E. N. Vidal†—I would like to know how you arrived at that twelve hour time for form removal and if that was changed in accordance with the different conditions found in the rock?

George Spann‡—On that particular section of the tunnel, the contractor placed the lining with the day shift. Usually the placing of the concrete was completed by about two o'clock in the afternoon. That meant that the day shift still had two hours before their shift ended. Those two hours were spent in cleaning out the cars and the pneumatic placers and having everything in shipshape for the next day. On the following shift, no work was done on that section of the sidewalls and arch, but on the shift after that, which would be the shift beginning at midnight and extending to 8 A. M. the next morning, the workmen would first open up the doors in the forms and start loosening the forms at the end of the section where the concrete was first placed. Working toward the end where the concrete had been placed last, this operation usually consumed about two hours and was completed at about two o'clock in the morning, which is twelve hours from two in the afternoon when they stopped placing the concrete lining. Now I assume from your question that you anticipate that that time would vary, owing to the amount of breakage in the tunnel excavation, which of course would vary the amount of concrete.

*A. C. I. JOURNAL, Feb. 1941, *Proceedings* V. 37, p. 325.

†U. S. Bureau of Reclamation, Custom House, Denver, Colo.

‡Assistant to the Chief Engineer, Board of Water Supply of New York, who presented the paper and led the discussion.

That is true, it does take longer to place concrete in a section where the over breakage was large than where the breakage was only to the normal line. However the difference in the time involved may have brought the finishing time for the placing one hour sooner or one hour later. The time could also vary on the other end of the work in stripping the forms and moving them ahead. The amount of time consumed by the entire operation was ample to maintain the 12-hour stripping time, on this section of the tunnel.

Mr. Vidal—I meant in case there was loose ground, did you have to wait longer before forms were stripped and was the strength of the concrete taken into consideration in regard to the twelve hours? Were you sure there was sufficient strength there before forms were stripped?

Mr. Spann—Your question about going through poor ground in relation to the stripping time is one that I did not have in mind when speaking of that particular operation. You must bear in mind that when a bad section of ground is encountered, the construction work in that area is quite different from that on the remainder of the job, In one case of such a section of bad ground, the contractor skipped that portion while he went through with his regular lining operation and came back and finished it later. In other words, I would not want you to interpret my statements about the general tunnel lining method as applying to special sections through bad ground. As to the strength of the concrete at the time of stripping the forms, that is something that can be seen as the work advances. We know from experience that twelve hours is ample, that there is no difficulty involved at all when twelve hours are permitted to elapse. However, where the stripping time is less, then it is customary first to open the doors in the forms and see the condition of the concrete at that point before loosening the forms themselves.

*Walter H. Wheeler**—Did you have water to contend with? If so, how was that handled?

Mr. Spann—Yes, we encountered water in many places in driving the tunnel. Of course, as the excavation advanced, any concentrated inflows were grouted off, or at least a very serious attempt was made to grout them off before excavation of that portion of the tunnel was continued. However, we still had water coming in at places, even with these precautions. Before placing the concrete lining on any portion of the periphery of the tunnel where water is coming in, sheet steel pans are first placed against the rock through which the water is entering. These pans are placed by the close drilling of holes around the

*Designing and Consulting Engineer, Minneapolis, Minn.

water bearing area, the nailing of the pans to wooden plugs forced into the holes and the calking of the outer edges of the pans to the rock. A grout pipe is then led through the steel sheet pan to convey the water away from that particular place while the concrete is being placed. Subsequently, after the lining is finished, a connection is made to this pipe and grout is injected to seal up the cracks through which the water is entering. In connection with the control of water in the invert, although the panning takes care of water entering through the invert rock, there may still be water leaking in from above which will tend to collect in the place where concrete is about to be placed. Where water does collect for one reason or another, small pumps are used for its removal and a suction line is always available for draining the water. In addition an air hose is used to blow out the water and get rid of it to the greatest possible extent.

*O. W. Irwin**—What is the advantage in your use of square twisted bars?

Mr. Spann—That again refers to a special section of lining; I think I will not answer it. It is a little outside of the scope of the paper in that it is a question on the particular type of bar used in special sections of the tunnel. The specifications do require square twisted bars in all of the major portions of the work. As it is somewhat outside of the field I intended to cover, I think I should not voice an opinion.

J. C. Pearson†—There is one other question I would like to ask Mr. Spann: When you place the concrete through the trap door, what assurance have you that it fills the space above the steel lining?

Mr. Spann—In the special section of the tunnel where steel interlining was used, the outer concrete lining was placed by means of a pneumatic placer through 6-in. standpipes attached to the upper side of the plates of the steel interlining. Of course holes were cut through the plates so that the concrete could be forced up through the six-inch standpipes. These standpipes were placed at intervals of about 20 ft., each having at its upper end a ninety degree elbow which was faced toward the bulkhead or concrete previously placed. The concrete placer was connected to the pipes successively and concrete was forced into each pipe until it showed up through the succeeding pipe, indicating that that part had been filled. This method was continued throughout the whole section until the entire space around the outside of the steel interlining was filled with concrete.

Lewis H. Tuthill‡—Many engineers have labored in the belief that concrete containing six sacks of cement per cubic yard is an excellent

*Carnegie-Illinois Steel Co., Pittsburgh, Pa.
†Director of Research, Lehigh Portland Cement Co., Allentown, Pa.
‡Denver. Mr. Tuthill's convention comments are here supplemented by written discussion.

concrete of ample strength and quality for long, efficient service in water tunnel linings. In view of this fact, there appears to be justification for some comment on the arbitrary use of one-third more than that cement content, or a total of eight sacks of cement per cubic yard, in the concrete for the Delaware aqueduct. The authors state that this "high cement content was adopted to insure concrete which should be as nearly watertight, dense, and insoluble as it is practically possible to attain." On the basis of current information relative to temperature rise from heat of hydration, thermal volume change as a cause of shrinkage cracking, density, and permeability, it does not appear that money is well spent for the extra two sacks of cement unless the extra 20 per cent, perhaps, of ultimate compressive strength is worth this premium. The philosophy that watertightness and durability can be obtained to a highly effective degree only by the use of such abnormal cement content reminds us that "it doesn't take an engineer to make it strong enough."

Tests of permeability* and experience clearly show that, for well placed concrete *without cracks*, there is inappreciable improvement in resistance to permeability when the water-cement ratio is reduced below 0.60 by weight. For average aggregates, 6-sack tunnel-lining concrete of 5-in. slump can be made with water-cement ratios not exceeding 0.60.

It is in connection with cracking that high cement content of ordinary or modified composition becomes serious. With six sacks of cement a 40-degree rise in temperature of the lining is common. With a third more cement at least a fourth greater temperature rise may be expected, thus adding at least ten degrees to the inevitable temperature drop. Resulting cracks will be proportionally wider or more frequent, and leakage through the wider cracks will be more than proportionally increased. Therefore, when such a surplus of cement is to be used, it would seem that the most appropriate selection of type would have been a type which exhibits minimum heat of hydration, either low-heat portland or a proven portland-pozzolan cement. With either of these cements, the same or better ultimate strength would have been obtained, other properties would have been comparable, and 9- to 12-hour strength for form stripping would have been sufficient, but the temperature rise of the concrete would have been less than with an ordinary amount of the type of cement used. If used, low-heat cement or clinker for portland-pozzolan cement should, of course, be made under the advanced regulations of manufacture promulgated by Mr. Merriman.

*Recently exemplified by Fig. 3, "Properties of Concrete Mixes," by Blanks, Vidal, Price, and Russell, A. C. I. JOURNAL April 1940; *Proceedings* V. 36, p. 448.

Mr. Spann—It is true that the cement content in the entire lining of the tunnel is two barrels or eight sacks per cu. yd. of placed concrete. These tunnels are in a place, of course, where no one may see the condition of the lining for many, many years. It may be that a lesser amount of cement will serve the purpose, but the extra quantity of cement used is considered to be a good insurance against possible defects in the lining and also it is considered that the extra amount of cement makes it possible to obtain a more compact or denser lining than may be possible with a lower cement content. It gives us a greater leeway for any unusual situation either in the nature of the ground that is passed through or because of the method used, so that the assurance is better that we are going to have a permanent, dense and durable lining for the aqueduct. As far as the strengths are concerned, 6 x 12-in. test cylinders are made as the work advances; tested at 28 days and show compressive strengths of well over 5000 psi. In connection with the cylinders and the tests that we have made upon them, we had occasion to remove some cores of concrete from the lining at places where the lining appeared to be poor and not up to the standard generally attained. These cylinders were cut and tested for compressive strength and the results showed that the concrete had a compressive strength of 5000 to 6000 psi.

I cannot say that we have had any undue amount of trouble from cracking. Of course the concrete does have cracks due to shrinkage; we have to expect those, especially when the concrete is placed over long stretches of tunnel in continuous pours. I am sorry that I haven't any actual count or measurements on the cracks, in fact I have not had much opportunity of seeing the concrete that has been placed after it had a good chance to crack in the tunnel. However, we expect some cracking and we also expect to seal these cracks when we go through with our grouting operations after the concrete has been given a chance to develop shrinkage cracks. The entire lining is grouted by forcing grout through pipes in the lining, and we believe that what cracks there are will be taken care of in this manner.

*Arthur A. Levison**—I would like to ask Mr. Spann what his experience was with the chuting of concrete, aggregates, and cement through these vertical pipes from ground surface down to the tunnel. The distance, I understand, runs from 600 to 800 or 1000 ft.; was there any segregation of these materials? How were the materials handled at the bottom of the pipe chute? How was the air suction developed by the dropping of these materials in the pipe chute taken care of?

*Blaw-Knox Co., Pittsburgh, Pa.

Mr. Spann—Where mixed concrete was dropped down the pipe, it was dropped in a set quantity, at one time, usually sufficient to fill the car or bucket in which it was transported through the tunnel. It may have segregated, but the fact that all such concrete was agitated before being finally placed in position nullifies any segregation that may have occurred in dropping it down the pipe. No problem as to suction was encountered in dropping the mixed concrete but it was met in dropping dry cement to be mixed in the tunnel. The difficulty was overcome by connecting a small pipe to the receiving hopper at the bottom and running it all the way up to the top.

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Expediting Construction on the Pennsylvania Turnpike*

By H. Hershey Miller†

SYNOPSIS

The Engineers of the Turnpike Commission were confronted with the task of placing more than four million square yards of nine inch uniform reinforced concrete pavement in two and one-half months.

The pavement consisted of four lanes, each lane twelve feet wide and one hundred and sixty miles long.

The analysis of the problem and the steps taken to solve it are the subject matter of this paper.

INTRODUCTION

January 1, 1940, the Pennsylvania Turnpike had emerged from the "dream" stage and had entered its last and most important phase, the placing of 4,000,000 square yards of concrete pavement.

Ninety per cent of the structures had been completed and the grading had progressed until only about 2,000,000 cubic yards out of a total of 24,000,000 cubic yards remained to be completed. Progress on the seven tunnels indicated that the last tunnel would be holed through before the middle of April.

Suppose that the following advertisement were to appear: WANTED: 2,028,585 bbl. of cement; 1,391,361 tons of stone; 843,877 tons of sand; 14,785 tons of steel. Materials must be delivered to 40 different destinations, many from 15 to 20 miles from nearest railroad. Material must be delivered and available for use between April 15, 1940 and July 1, 1940. It will require daily deliveries of 191 cars of cement; 242 cars of stone; 779 cars of sand; and 18 cars of steel; in addition, sufficient trucks to transport 1,000,000 tons of stone direct from

*Received by the Institute Nov. 20, 1940.
†Resident Engineer Pennsylvania Sand and Gravel Producers Assn., Steelton, Pa., formerly Testing Engineer Penna. Turnpike Commission.

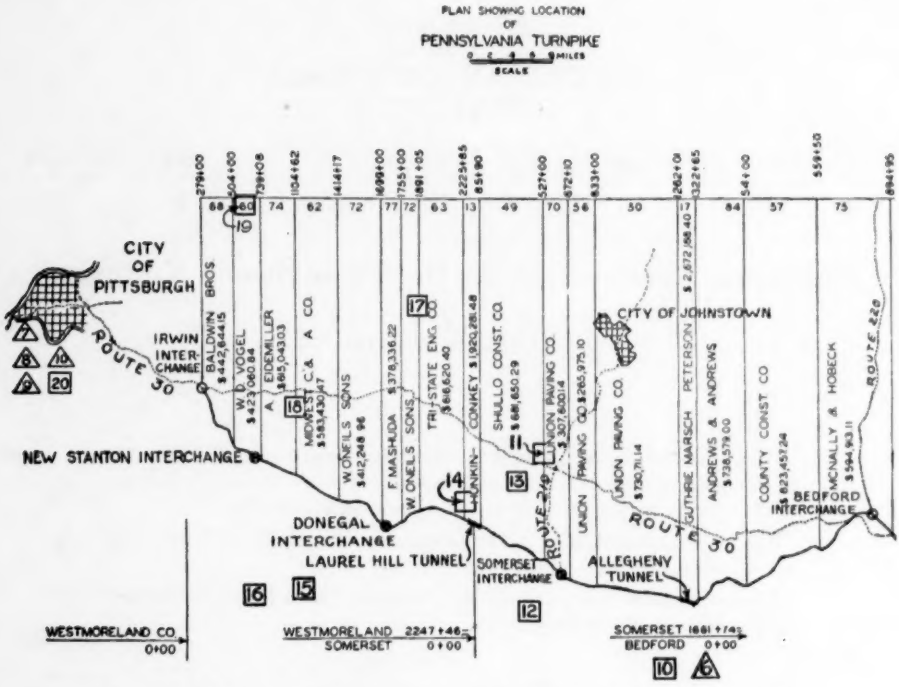

FIG. 1—MAP OF PENNSYLVANIA TURNPIKE (MAP CONTINUED OPPOSITE PAGE)

Twenty-nine paving projects and bid prices, six tunnel jobs, and sources of aggregates—coarse, shown by numbered rectangles, fine aggregate by numbered triangles—referenced to the following tabulation of producers.

Coarse Aggregate Producers		Sand Producers
1. Bethlehem Mines	11. J. A. McNeil	1. Susquehanna S. & G. Co.
2. J. F. Sours	12. Romesburg Stone Co.	2. J. A. Eck & Co.
3. Shippensburg St. Co.	13. Laurel Ridge	3. Lycoming Silica Sd. Co.
4. Concrete Mat. Co.	14. Hunkin-Conkey	4. Arundel Corp.
5. Contractors Ser. Co.	15. Vang Const. Co.	5. Campbell & Son
6. H. B. Mellott	16. Newcastle L. & S. Co.	6. A. H. Smith
7. N. Enterprise Sd. Co.	17. Graystone Quarry	7. J. K. Davison
8. Concrete Mat. Co.	18. A. Eidemiller	8. Dravo Corp.
9. Gildea Co.	19. Pittsburgh L. & S. Co.	9. Iron City Sd. Co.
10. Cumberland C. & S. Co.	20. J. K. Davison	10. McCready-Rogers

quarries to batching plants. Sufficient equipment and personnel to batch, transport, mix, place, and finish 4,005,203 square yards of 9 inch uniform concrete pavement. Such an advertisement did appear, not in the aggregate, but covering the twenty-nine individual paving projects.

Fig. 1 is a map showing the locations of the twenty-nine paving projects and the six tunnel projects, together with bid price for each

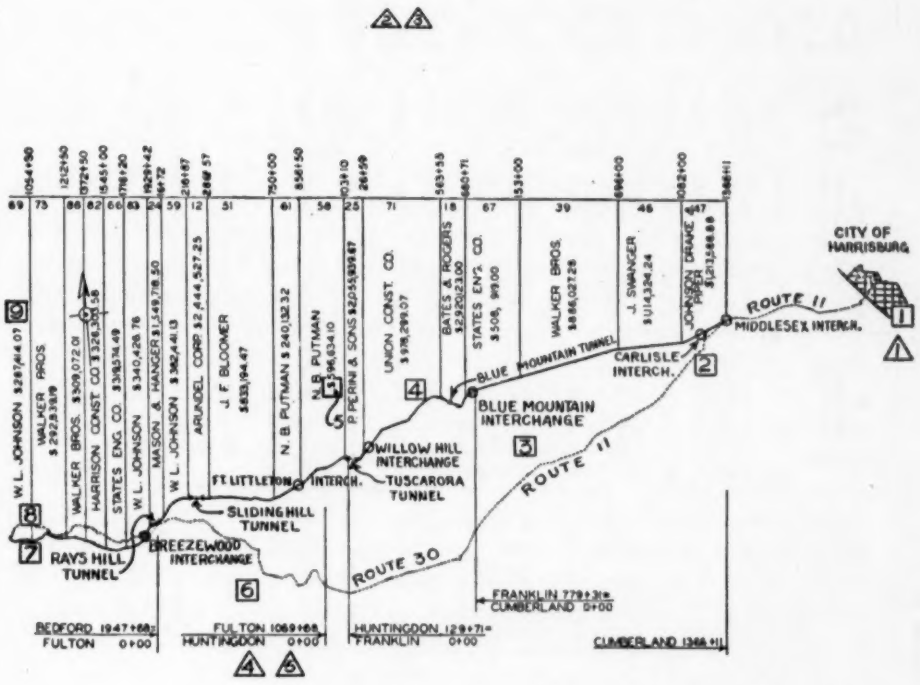

FIG. 1—CONTINUED FROM OPPOSITE PAGE

project; locations of fine and coarse aggregate sources of materials for the concrete pavement.

This map is presented since it gives the geographic location of the Turnpike and indicates the difficult terrain that had to be traversed in its construction.

MATERIAL REQUIREMENTS

The amounts of material required for the construction of the pavement are impressive. When a survey of material sources and production was made, together with availability of transportation, loading sidings, and storage siding facilities, it became appalling. Fortunately coarse aggregates for 23 contracts and steel for 21 contracts was transported direct from source to the projects by truck.

All cement was transported by railroad and all fine aggregate was shipped by rail except some small amounts trucked in the early spring.

Table 1 lists the material requirements for each project. Our first problem was to ascertain from the contractors the sources from which they intended to secure materials. After these were tabulated

TABLE 1—MATERIAL SOURCES

Cont. No.	Material Sources					Barrels Cement	Tons Coarse Aggregate	Tons Fine Aggregate	Ton Steel
	Cement Sources	Coarse Aggregate	Fine Aggregate	Reinforcing Fabric	Transverse Jt. Assembly				
47	Atlas, Allentown, Alpha, Penn-Dixie, Lehigh	J. F. Sours	Arundel Susq. Sd. & Gl.	"A. & C." Am. S. & W.	K Beth. Steel	61000	40000	26000	600
46	Allentown, Alpha, Penn-Dixie, Lawrence, N. American, Keystone	Beth. Mines	Arundel Campbell	"A. & C." Am. S. & W.	K Beth. Steel	76400	44860	35080	550
39	Lehigh, Penn-Dixie, Medusa	Shippensb'g St.	Arundel Campbell	Beth. Steel "D"	K Beth. Steel	120000	82000	54000	1000
67	Lehigh	Shippensb'g St.	Arundel	"A. & C." Am. S. & W.	M 5 way Jt. Co.	60000	42000	24000	470
71	Penn-Dixie	Conc. M. & C.	Arundel	Laclede "A & C"	M 5 way Jt. Co.	112320	78237	40865	800
58	Keystone	Cont. Ser. Co.	J. A. Eck	Pitts. St. "A & C"	D Pitts. St. Co.	63500	43000	25500	430
61	Keystone	Cont. Ser. Co.	J. A. Eck	Pitts. St. "A & C"	D Pitts. St. Co.	25300	19700	10000	150
51	Bessemer, Atlas, Medusa, Penn-Dixie	Mellott	Arundel	Pitts. St. "A & C"	D Pitts. St. Co.	86000	60000	31000	466
59	Medusa	Mellott	Lycoming	Truscon St. "D"	K Beth. Steel	41415	27874	16592	308
83	Medusa	Mellott N. Enterprise	Lycoming	Wheeling "ACE"	K Beth. Steel	35000	30000	15000	250
66	Lehigh	N. Enterprise	J. A. Eck	Pitts. St. "A & C"	M 5 way Jt. Co.	37000	26000	15000	300
82	Universal-Atlas	N. Enterprise	Lycoming	Am. S. & W. "A"	N 5 way Jt. Co.	37000	25000	15000	380
88	Alpha	N. Enterprise	Lycoming	Beth. Steel "D"	K Beth. Steel	34500	25100	14500	215
73	Alpha	N. Enterprise	J. A. Eck	Beth. Steel "D"	K Beth. Steel	33300	23600	13700	220
69	Medusa	Conc. M. & C.	Lycoming	Truscon St. "D"	K Beth. Steel	30300	21140	12600	244
75	West Penn	Conc. M. & C.	Lycoming	Pitts. St. "A & C"	M 5 way Jt. Co.	70000	60000	30000	700
57	Medusa, W. Penn, Standard, Penn-Dixie, Green Bag	Conc. M. & C. Gildea	J. A. Eck	Beth. Steel "D"	K Beth. Steel	108000	65000	44000	600
84	Alpha	Conc. M. & C. Romesburg St.	A. H. Smith	Beth. Steel "D"	K Beth. Steel	85000	52000	33000	500
50	National, Lone Star, Standard	McNeil Cumberland C. & S. Co.	Iron City	Beth. Steel "D"	K Beth. Steel	95000	63000	40500	600
56	Keystone, Standard	McNeil, Ferner Vang	McCrady	Pitts. St. Co. "A"	D Pitts. St.	26500	18500	11000	170
70	Lehigh	McNeil, Ferner Vang	Iron City McCrady	Pitts. St. Co. "A"	D Pitts. St.	34000	28000	14000	270
49	Universal, Atlas, West Penn, Bessemer, Pitts. Plate Glass Co.	New Castle	McCrady	Truscon St. "D"	K Beth. Steel	88200	55000	33000	540
63	Medusa	Vang	Iron City	Pitts. "A. & C."	D Pitts. St.	68000	42000	28000	575
72	Universal-Atlas	Hunkin-Conkey	Iron City	Cons. Ex. Metal	K Beth. Steel	49800	36300	20400	400
77	Universal-Atlas	Hunkin-Conkey	Iron City	Cons. Ex. Metal	K Beth. Steel	16150	10550	6500	75
62	Bessemer	Graystone	McCrady	Am. S. & W. "A"	M 5 way Jt. Co.	63000	45000	27000	410
74	Universal-Atlas	Pitts. L. & S. Co. Eidemiller	Iron City McCrady	Am. S. & W. "A"	D Doweld Inc.	73000	48000	32000	550
60	Green Bag	J. K. Davidson	J. K. Davidson	Am.S. & W. "A"	M 5 way Jt. Co.	50000	30000	20000	600
68	W. Penn, Universal-Atlas	J. K. Davidson	J. K. Davidson	Wheeling "ACE"	M 5 way Jt. Co.	55000	34000	22000	350

and the total material required from each source calculated, an inspection was made of each source and a detailed report as to production was obtained. When these figures were analyzed, it was demonstrated that a number of sources did not have the capacity to supply the required amounts. Each producer was furnished with our analysis and in some cases by voluntary agreement the contractors were induced to place orders with other producers. In other cases producers took the necessary steps to increase production, either by increasing capacity or increasing working hours so that an adequate supply would be assured.

Cement. Our specifications required that all cement should be bin tested and that the results of 28-day strength be available before shipments could be made.

The purpose of these requirements was threefold; first, it assured ample time for complete tests, such as, autoclave, soundness, surface area, chemical analysis, and strength; second, it precluded the possibility of hot cement and the attendant finishing problems; and third, full information as to the quantity of cement in storage at each plant was known in advance so that a switch could be made when it became apparent that certain companies could not fulfill their obligations. These precautions, while not meeting the approval of all companies, proved of value when strikes threatened to tie up a number of contracts.

Fine aggregate. Specifications required that fine aggregate be stocked twelve hours prior to use. This provided ample time to conduct field tests to determine if material complied with specification requirements. It also resulted in more uniform moisture content which is a vital factor in the control of the water content of the concrete. An adequate supply of fine aggregate was stocked on each contract so that delays in shipment would not retard production. Contractors were encouraged to stock materials well in advance of use by being paid estimates on materials stocked. This precaution not only assisted producers but assured a continuous supply during the peak production periods.

Coarse aggregate. As previously stated, coarse aggregate was transported from source to batch plant by truck on 23 of the projects. This presented a difficult problem at times since non-uniformity of moisture content made concrete control most difficult. However, by mutual agreement, contractors cooperated and coarse aggregate was, except in emergencies, stocked prior to use. As in the case of fine aggregate, the contractors were paid for coarse aggregate stocked prior to use. Coarse aggregate was stocked and batched in two

different sizes to avoid segregation and to permit adjustments in the mix to improve workability.

Reinforcing steel. With the exception of eight contracts, all reinforcing steel was furnished direct to the contracts by truck, being spotted along the project as required. This steel was furnished from accepted stock stored at warehouses located within trucking distance from the construction.

TRANSPORTATION

An early analysis indicated that transportation facilities, especially sidings, would be taxed to capacity to supply the requirements for the Turnpike. Switching facilities were not adequate to transport the daily requirements and remove the empty equipment.

The number of bulk cement cars available indicated that a sufficient amount of this type of equipment would not be available. An early survey showed 1584 bulk cement cars in service east of Chicago and 305 cars under construction.

A study of round trip movement time, including loading, transporting, unloading, and return, indicated an average round trip time of six days. This would require a total of 1146 cars to service the Turnpike contracts. Since a large number of bulk cement cars were in other service, it appeared that less than 700 cars, including those under construction, would be available for Turnpike use.

Aggregate requirements totalling 1000 cars per day presented another problem for transportation, equipment, and siding facilities.

Representatives of the Pennsylvania R. R., Baltimore & Ohio R. R., Huntingdon and Broad Top R. R., and East Broad Top R. R. were called into conference and our problem was presented.

Finally, by a fine spirit of cooperation, it was agreed to make up full length cement freight trains from the Lehigh District and Pittsburgh District, and transport them direct to the projects without reclassifying at freight terminals.

The railroad companies also agreed to increase switching facilities so that twenty-four hour switching schedules would be available to transport material from main lines and storage yards to spurs and unloading sidings and to return empties. By this method, round trips for cement cars was reduced from six days to approximately three and one-half days.

Precautions had been taken relative to bulk cement cars by informing contractors that provisions would have to be made for unloading cement from box cars in the event of a shortage of bulk cement cars.

Materials not immediately required were stored on sidings or storage yards sometimes at considerable distance from the projects and switched to the contracts as required.

Trucks. Even when materials were delivered to rail heads it required, in many cases, long truck hauls to batch plants. Large areas of Franklin, Bedford, and Somerset Counties traversed by the Turnpike were not near any railroads and Fulton County has no railroads within its borders.

These trucking operations had to be on a 24-hour schedule, and provision had to be made to maintain these schedules so that paving progress would not be delayed.

FIG. 2 AND 3—CONSTRUCTION VIEWS PENNSYLVANIA TURNPIKE

(2) At the top, two 27E pavers on opposite sides of roadway placing bottom and top courses in same lane; tank truck water supply. (3) bottom, placing and finishing operations in successive stages.

TABLE 2—MINIMUM REQUIREMENTS, 44 PAVING DAYS

Total Sq. Yds.	Daily Sq. Yds.	Using 27E		Using 34E	
		Pavers	Shifts	Pavers	Shifts
75,000	1705	1	2—7 Hr.	1	1—8 Hr.
100,000	2273	1	2—8 Hr.	1	2—7 Hr.
125,000	2841	2	2—6 Hr.	1	2—7 Hr.
150,000	3409	2	2—7 Hr.	1	2—8 Hr.
175,000	3977	2	2—8 Hr.	1	2—8 Hr.
200,000	4545	2	3—7 Hr.	1	3—7 Hr.
225,000	5113	3	2—7 Hr.	2	2—6 Hr.
250,000	5681	3	2—8 Hr.	2	2—7 Hr.
275,000	6249	4	2—7 Hr.	2	2—7 Hr.
300,000	6817	4	2—7 Hr.	2	2—8 Hr.

To complicate the trucking operations further, it became necessary for the Pennsylvania Department of Highways to impose weight restrictions on some of the lighter type macadams during the early spring. Fortunately these restrictions were removed as soon as the frost had left the subgrade, and the surfaces had been repaired.

PAVING SCHEDULES

After the Turnpike Engineers had taken every precaution possible to assure an adequate, continuous supply of acceptable material; it was necessary to analyze each contract and set up schedules that would assure completion within time limit set for each contract.

From data secured from the Pennsylvania Department of Highways and from Weather Bureau Records, it was determined that approximately 44 days would be available for paving from April 15 to July 1.

Based on records made available to the Commission by the Pennsylvania Department of Highways and from the records of six turnpike projects where paving was placed during the fall of 1939, it was possible to determine an expected job efficiency rating.

The mixing time per batch required by specification was one and one-quarter minutes. By adding charging time and discharging time it was assumed that 40 batches per hour was the maximum for single drum pavers and 60 batches per hour for dual drum pavers.

The average job efficiency, that is, the percentage of the maximum number of possible batches was found to be 72.7 per cent based on Highway Department data and Turnpike records. Thus with 100 per cent efficiency, we could expect 319.8 square yards of pavement per hour with a dual drum 34E paver and 168.4 square yards with a single drum 27E paver. Applying the 72.7 per cent job efficiency record, we obtained an expected 232.5 square yards for dual drum 34E and 122.4 square yards for a single drum 27E mixer. From these data Table 2 was prepared.

Table 3 lists the actual equipment used on each contract and it will be noted that considerable latitude was granted the contractors in determining the amount and type of equipment to be used.

The table of minimum mixer requirements was furnished as a guide and the engineers were willing to accept any combination that would maintain the daily production schedule.

Needless to say the batching, hauling and finishing equipment requirements were studied as closely as the paving equipment to avoid bottlenecks and other delays that are usually chargeable to one or more of these factors.

The major causes of delay in paving operations encountered on the Turnpike projects were: breakdowns, 33.8 per cent; trucks, 15.2 per cent; forms, 2.9 per cent; grade, 2.6 per cent; miscellaneous, 13.6 per cent; time, 31.9 per cent. The percentage charged against "time" is that lost through the 40-hour week limitations. The percentage charged against miscellaneous was due to lack of materials, water, delays in finishing, etc. The percentages do not include delays for which the contractor is not responsible, such as, weather, moving over gaps, moving equipment, etc.

By good supervision and management, it is possible to eliminate all delays except those created by breakdowns, and these can be kept at a minimum by maintaining a repair organization and by keeping spare parts for the most common breakages.

TYPICAL PAVING CONTRACTS

34E Paver. The typical paving contract used a 34E dual drum paver, one 3-compartment aggregate batcher, one cement batcher, one concrete spreader, two transverse finishing machines, and one longitudinal finishing machine.

The batch plant was located as close to the work as siding facilities could be obtained. Since in a number of cases railroad facilities were not available, bulkhauling to batch plant was necessary. One crane was usually employed for unloading and two cranes for charging batch bins. Ten to thirty trucks of two-batch capacity were used for transporting batches dependent on length of haul.

Subgraders, form tampers, and mechanical pin drivers were used for preparing grade and setting forms.

Pavers operated outside the forms and expansion joints were set well in advance of paving. Expansion joints were required at intervals not exceeding 77 feet and in the usual paving operation, concrete was placed for the bottom course, struck off two inches below the finished surface by machine and the reinforcement placed. The

TABLE 3—EQUIPMENT SOURCES

Cont. No.	Batching		Mixing		Placing and Finishing			Curing	Price Per Sq. Yd.	Sq. Yds.	Shifts
	Aggregate	Cement	Make	Operation	Spreading	Transverse	Longitudinal				
47	Johnson	Butler	34 Koehring	12′ Width	1 Jaeger	2 Jaeger	1 Koehring	Paper	2.70	142052	2 8 Hr.
46	2 Blaw Knox	Butler	34 Koehring	12′ Width	1 Jaeger	1 Jaeger	1 Koehring	Paper	2.63	190047	2 7 Hr.
39	Blaw Knox	Blaw Knox	34 Rex	12′ Width	1 Jaeger	2 Blaw Knox	1 Koehring	Mats	2.35	288290	2 8 Hr.
67	Blaw Knox	Blaw Knox	34 Koehring	12′ Width	1 Jaeger	2 Jaeger	1 Koehring	Paper	2.84	145966	2 7 Hr.
71	2 Butler	2 Blaw Knox*	34 Rex	2 12′ Width	2 Jaeger	2 Heltzel	2 Koehring	Paper	2.94	294032	3 7 Hr.
	Blaw Knox		27 Rex (2)	2 12′ Width	2 Jaeger	2 Heltzel	2 Koehring	Paper			2 8 Hr.
58	Heltzel	Heltzel	34 Rex	12′ Width	1 Jaeger	1 Heltzel	1 Koehring	Paper	2.96	161726	2 8 Hr.
61	Heltzel	Blaw Knox	27 Rex	12′ Width	1 Jaeger	1 Heltzel	None	Paper	3.01	64982	2 7 Hr.
51	Heltzel	Acme**	34 Koehring	24′ Width	1 Heltzel	1 Heltzel	1 Koehring	Paper	2.92	196598	3 7 Hr.
59	Blaw Knox	Blaw Knox	34 Koehring	12′ Width	1 Jeager	2 Jaeger	1 Koehring	Paper	2.98	106143	2 8 Hr.
83	Blaw Knox	Heltzel	2 34 Koehring	12′ Width Tandem	1 Jaeger	2 Jaeger	1 Koehring	Paper	3.07	87078	2 8 Hr.
66	Blaw Knox	Blaw Knox	27 Ransome dual	12′ Width	1 Jaeger	1 Jaeger	1 Koehring	Paper	3.14	90675	2 8 Hr.
82	Blaw Knox	Butler	27 Koehring	12′ Width	1 Jaeger	2 Jaeger	1 Koehring	Paper	3.04	92208	2 7 Hr.
88	Blaw Knox	Butler	34 Rex	12′ Width	1 Blaw Knox	2 Blaw Knox	1 Koehring	Mats	2.84	81301	2 8 Hr.
73	Blaw Knox	Butler	34 Rex	12′ Width	1 Blaw Knox	2 Blaw Knox	1 Koehring	Mats	3.05	84469	2 8 Hr.
69	Heltzel	Heltzel	2 27 Koehring 1 34 Koehring	12′ Width	1 Jaeger	2 Jaeger	1 Koehring	Paper	3.04	85385	2 7 Hr.
75	2 Butler	2 Butler	34 Koehring 27 Rex	12′ Width Tandem	1 Jaeger	2 Jaeger	1 Koehring	Paper	2.85	183349	2 7 Hr.
57	2 Blaw Knox	2 Blaw Knox	34 Rex 27 Rex	12′ Width Tandem	1 Jaeger 1 Blaw Knox	2 Blaw Knox	1 Koehring	Paper	2.70	270469	2 8 Hr.
84	2 BlawKnox	2 Blaw Knox	27 Ransom dual 27 Ransom	12′ Width Tandem	1 Blaw Knox	2 Blaw Knox	1 Koehring	Paper	3.10	209013	2 8 Hr.
50	Blaw Knox	1 Blaw Knox	2 27 Ransome	12′ Width Tandem	1 Jaeger	1 Ord 1 Blaw Knox	None	Paper	2.78	228011	2 8 Hr.
56	1 Blaw Knox 1 Acme	Johnson	27 Ransom 34 Koehring	12′ Width Tandem	1 Jaeger	1 Ord 1 Blaw Knox	1 Koehring	Paper	2.70	43828	2 8 Hr.
70	1 Blaw Knox 1 Acme	Johnson	27 Ransom 34 Koehring	12′ Width Tandem	1 Jaeger	1 Ord 1 Blaw Knox	1 Koehring	Paper	3.02	85706	2 8 Hr.
49	Blaw Knox	Blaw Knox	34 Rex	12′ Width	1 Jaeger	2 Blaw Knox	None	Paper	2.61	178812	2 7 Hr.
63	2 Blaw Knox	Blaw Knox	27 Koehring 34 Koehring	12′ Width Tandem	1 Jaeger	2 Jaeger	1 Koehring	Paper	3.03	165168	2 7 Hr.
72	1 Blaw Knox	Blaw Knox	27 Rex 34 Rex	12′ Width Tandem	1 Jaeger	1 Jaeger	1 Koehring	Paper	3.04	121150	2 7 Hr.
77	1 Blaw Knox	Blaw Knox	27 Rex 34 Rex	12′ Width Tandem	1 Jaeger	1 Jaeger	1 Koehring	Paper	3.71	36610	2 7 Hr.
62	Blaw Knox	Butler	27 Foote 34 Koehring	12′ Width Tandem	1 Blaw Knox	1 Ord 1 Blaw Knox	None	Mats	3.16	164832	2 7 Hr.
74	Johnson	Johnson	27 Koehring 34 Koehring	12′ Width Tandem	1 Jaeger	2 Jaeger	1 Koehring	Paper	2.93	192414	2 7 Hr.
60	2 Blaw Knox	Blaw Knox**	27 Ransom dual 27 Rex	24′ Width Tandem	1 Heltzel	1 Heltzel	1 Koehring	Paper	2.73	127800	2 7 Hr.
68	1 Johnson 1 Erie	Heltzel	34 Koehring	12′ Width	1 Lakewood	2 Lakewood	None	Paper	2.78	133501	2 7 Hr.

*Operating 3 pavers and 4 finishing operations. **Using Heltzel Flex-plane, paving 24′ width.

paver then returned and placed the top course which was spread by machine and then finished by one pass of each transverse finishing machine followed by the longitudinal finishing machine.

Final finish, edging, and transverse joints were completed by hand. The final operation was a burlap drag used to give a rough non-skid texture to the surface.

Curing on the typical job included wet burlap or mats for 24 hours followed by impervious paper for 48 hours.

All pavement was straight-edged with a 10-foot to 16-foot straightedge while still in a plastic state, and low or high spots were corrected. After the concrete had hardened, it was again straight-edged and high points were corrected by use of approved grinding tools so that deviations in excess of $\frac{1}{8}$ in. in sixteen feet were corrected.

Tandem operation 34E and 27E pavers. The typical tandem operation consists of one 34E dual drum paver placing bottom course and one 27E single drum paver placing the material on the surface after the reinforcement is placed. Two aggregate bins are used so that batch weights need not be reset, and trucks are marked to indicate the size of batch.

The pavers operate at a distance of approximately 75 feet and the full efficiency of the 27E paver is rarely utilized. It requires nice correlation in operation of mixers and timing of truck for most efficient operation. Production indicates about 25 per cent increase over the 34E paver used alone.

Some of the advantages of tandem operation include: increased production with the same finishing equipment, full production from the 34E pavers, since no time is lost in moving back to place surface course, supervision of paving operation under one head, in the event of breakdown of one paver, operations while curtailed, are not entirely suspended.

Other operations in tandem paving follow the same pattern as for single paver construction.

PERFORMANCE

Fig. 4 shows the paving schedule and the actual performance. It will be noted that the actual production for the last half of April was considerably below schedule, caused by an unusually late spring, which delayed the starting date on all contracts.

It is interesting to note, however, that from May 1 to July 1, the actual progress was in excess of schedule and by July 7 all concrete pavement had been placed except some short gaps adjacent to the tunnels, where tunnel operations delayed the paving progress.

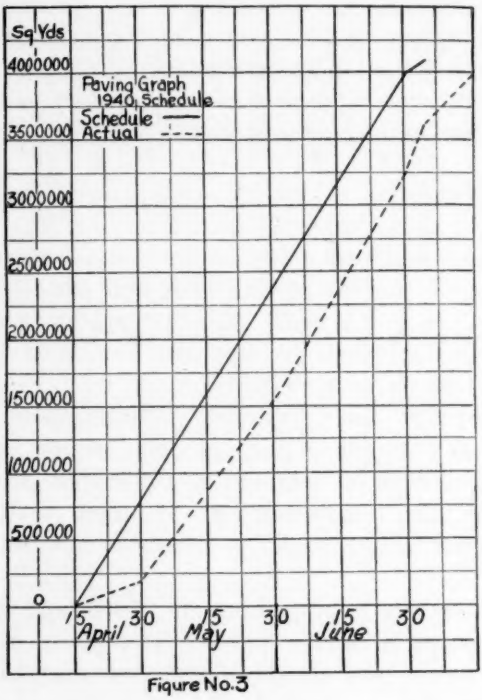

FIG. 4—SCHEDULE VS. PERFORMANCE

In looking over the record it is evident that construction projects, even on the scale of the Turnpike, involving a number of individual contractors, may, by proper coordination and cooperation, maintain a proposed schedule, provided the problem is carefully analyzed and a solution vigorously followed.

ACKNOWLEDGMENT

Special acknowledgments are due to S. W. Marshall, Chief Engineer of the Turnpike Commission, and to Rodger Stone, F. S. Poorman, J. D. Paul, and other engineers of the Turnpike for data supplied and valuable suggestions.

H. S. Mattimore and other engineers of the Pennsylvania Department of Highways supplied valuable information on production capacities and quality of material of producers of fine and coarse aggregate, as well as valuable data on probable paving days and probable production, which were used as a basis for setting up schedules for each contract.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by H. Hersey Miller:

Expediting Construction on the Pennsylvania Turnpike*

BY L. E. ANDREWS AND AUTHOR

BY MR. ANDREWS†

Mr. Miller's chronicle is a terse, concise report on the planning, team work and resourcefulness of the engineers, contractors, material producers, and transportation agencies involved in carrying forward to successful conclusion the nation's outstanding highway project at the record speed of more than 1,000,000 sq. yd. of concrete pavement per month.

The siding and switching requirements for the handling of more than 1200 carloads of materials daily and the maintenance of side roads for the passage of thousands of truckloads of stone each day is an indication of the herculean nature of this undertaking.

That all of this effort was fully justified is now being demonstrated by the fact that traffic volume and service requirements are exceeding expectations. Truly, great credit is due to the Turnpike Commission and its staff of engineers recruited from the four corners of the country in planning this work so well.

Mr. Miller's paper is devoted to the concrete pavement construction. Specifications for this work employed the best thoughts in construction for the pavement and safety of operation for the vehicle.

For expediting the work, tandem operation of pavers was the general rule, the bottom course to steel elevation, two inches below the surface, being placed by 34E dual drum pavers, while the top two inches was placed by 27E single drum pavers. By this arrangement production was increased by about 25 per cent over the use of the 34E

*A. C. I. JOURNAL, Feb. 1941, *Proceedings* V. 37, p. 349.
†Regional Highway Engineer, Portland Cement Association, New York

paver alone. All pavers were loaded 10 per cent in excess of rated capacities. In addition, the average job used one concrete spreader, two transverse finishing machines, and one longitudinal finishing machine.

The pavement surface was scraped with heavy section, wooden straight edges to remove excess surface mortar. Final surface texture was produced by use of burlap drag.

Coarse aggregate was handled in two separated sizes proportioned for best workability and the percentage of fine aggregate to total aggregate was adjusted for best results without an excessive amount of mortar. Proportioning was on the basis of fixed absolute volume of aggregates per bag of cement and maximum water ratio. Exceedingly uniform concrete was obtained as was witnessed by observations at the time of construction and from cores taken from the finished job.

The control of concrete materials in the field resulted in supplying uniform concrete to the jobs and in this way contributed to the expeditious handling and completion of the work. Particular attention, however, is called to the fact that far-sighted planning was directed toward every feature and agency involved in order that all parts of a complicated procedure might fit together perfectly to complete this colossal program without delay.

AUTHOR'S CLOSURE

No one is in a better position to give an authoritative account of this project than Mr. Andrews. He covered the project on a number of occasions and brought with him engineers from other states to study procedure and methods.

It is unfortunate that extensions of this project are not now under construction, since a net work of similar roads would without a doubt prove to be a valuable asset to our present transportation system, for national defense.

A recent study of transportation in England published by the National Highway Users Conference under the title: "Don't Repeat British Blunder" compares England's slow truck movement due to lack of adequately planned highways and points to the extreme mobility achieved by the Germans over their network of through highways.

It is to be hoped that our nation will soon take steps to provide adequate highways designed for quick movement of vital defense materials and military forces. Such roads could be used without paralyzing normal traffic on the present highways and have a peacetime value in expediting through traffic. The fact that such highways as

is being demonstrated on the Pennsylvania Turnpike are self-liquidating should be a governing factor in the early resumption of construction on the proposed system.

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Concrete Control on the Pennsylvania Turnpike*

By I. L. Tyler†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The paper deals with problems encountered in controlling quality of paving concrete on a large project, The Pennsylvania Turnpike, constructed in record time. Procedures adopted for maintaining such control are briefly discussed.

Suggesting that uncontrolled mixing water has been the important factor in most of the recorded failures of concrete structures, the paper directs particular attention to the phase of construction having to do with its control. Data are presented which indicate that the actual amount of mixing water in concrete may be consiterably greater than is generally supposed. Effects of type of concrete placing equipment on quality of concrete and some effects of construction practices and procedures are considered. Suggestions are offered which may be of value in preparing paving specifications.

The Pennsylvania Turnpike is an express highway approximately 160 miles long with termini short distances from Harrisburg and Pittsburgh, Pa., respectively. About three-quarters of the route traverses mountainous or hilly country. Seven tunnels through the higher ridges permit moderate grades and excellent alignment through difficult terrain avoiding to an appreciable extent the maintenance difficulties which would be encountered at higher altitudes. Except through the tunnels and over major bridge structures the roadway consists of two 12-ft. lanes in each direction separated by a 10-ft. dividing strip and bordered by 10-ft. shoulders. The full width pavement is reinforced concrete of 9-in. uniform thickness having a joint spacing of 77 ft. Pavement in the tunnels, also of reinforced concrete, consists of two 11½-ft. lanes, 10 in. thick with 30-ft. joint

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†Formerly Materials Engineer, Pennsylvania Turnpike Commission, Harrisburg; now Research Engineer, Portland Cement Assn.

spacing. Speed of construction characterized the building of this ultramodern highway.

The necessity for extremely rapid construction introduced problems of controlling quality of materials which in a slower program, with more opportunity for consideration, might have been avoided. How well these problems have been solved can be answered with more assurance later.

The phase of Turnpike construction to be briefly discussed in this paper, that of concrete paving construction, was among those demanding the closest control of materials and procedures. The paving schedule contemplated the placing of about 4½ million square yards of 9-in. concrete pavement on 29 contracts, exclusive of tunnels. Circumstances finally dictated that most of this be placed during approximately 2½ months in the spring of 1940. Fortunately, one contractor was able to place most of his paving during the fall of 1939, and five others obtained starts in the late fall. This provided an excellent opportunity for studying contractors' equipment and operations, inspection requirements, and concrete control procedures well in advance of the highly concentrated program to follow in the spring. These studies and observations served as a basis for establishing control methods for use on the remainder of the paving construction and offered an opportunity for training a small nucleus of inspectors and key men. Data on concrete mixes, especially in regard to mixing water requirements made available by the early operations were of particular value later.

MATERIALS

Insofar as possible, standard specifications were used for all concrete materials following closely requirements adopted or proposed for use by the Pennsylvania Department of Highways. The necessity for obtaining a very large quantity of material in a very short time precluded any possibility of considering special materials for use on the project. The few variations from standard practice in the locality were included with some misgiving and only because they were considered highly desirable to the best interest of the project. The Pennsylvania Department of Highways Testing Laboratory handled all of the laboratory testing of cement and aggregates for Turnpike construction and it was due in no small way to the fullest cooperation of this organization that adequate supplies of suitable materials were made available without delay.

Cement specifications covered normal portland cement with additional requirements for fineness, autoclave soundness, and a provision

FIG. 1—PENNSYLVANIA TURNPIKE, OVERLOOKING EVERETT, PENNA.

for increasing time of initial set during hot weather. The last two requirements had been included in the specifications of the Pennsylvania Department of Highways for some time and created no problem. A minimum fineness corresponding to a specific surface of 1600 sq. cm. per gram (Turbidimeter) was added to improve uniformity in characteristics of fresh concrete.

Aggregate specifications covered 4-mesh sand and two sizes of coarse aggregate (designated as 2B and 3A), all meeting standard A. A. S. H. O. requirements for grading, except that 10 per cent of the sand was required to pass the 50-mesh sieve. Washed natural sand in combination with either crushed stone or gravel containing 40 per cent of crushed particles was used in all pavement concrete. A maximum loss of 10 per cent in five cycles of sodium sulfate was permitted for sand and coarse aggregates. Table 1 shows grading requirements for aggregates used in pavement concrete.

Aggregates meeting these grading specifications in the required amounts were obtained without difficulty. Predictions that it would be impossible to obtain sand with more than 10 per cent passing the 50-mesh sieve in such large quantities were evidently unfounded, but a study of sand production problems indicated that sand processing methods may be in for some revision in the not too distant future if grading requirements become more rigid or even if the present standard as indicated above is to continue.

Previous experience gained from observation of concrete used in almost every type of construction further verified by Turnpike results convince the writer that 15 to 20 per cent of most natural sands should pass the 50-mesh sieve and an appreciable quantity should also pass the 100-mesh. It was found that the 10 per cent requirement usually resulted in obtaining sand with at least 12 per cent passing the 50-mesh sieve and an appreciable amount finer than 100-mesh. The writer would strongly recommend a 15 percent minimum limit whenever possible. The importance to concrete quality of having sufficient quantities of material finer than the 50-mesh and 100-mesh sieves can hardly be over-emphasized.

Tests on mixing water sources were made in order to avoid the possibility of introducing deleterious materials into the concrete, particular care being observed in sampling and testing in mine water areas.

TABLE 1—GRADING EQUIPMENTS

Sieve Size	Per Cent Passing Square Sieves		
	Sand	2B	3A
2½-inch			100
2 -inch			90–100
1½-inch		100	35–70
1 -inch		90–100	0–15
½-inch		25–60	
4-mesh	95–100	0–10	
16-mesh	45–80		
50-mesh	10–30		
100-mesh	0–8		

CONCRETE MIXES

Specifications for paving concrete called for 3.12 cu. ft. absolute volume of aggregate for each 94-lb. sack of cement and a maximum water-cement ratio of 5½ gal. per sack for the normal mix containing both sizes of coarse aggregate. Limited variations in proportions of each aggregate size were provided so that adjustments in proportions could be made as characteristics of materials and operating conditions demanded. Concrete meeting these requirements contained approximately 1.55 bbl. of cement per cu. yd. A provision for increasing cement content was added to care for conditions under which the 5½ gal. per sack water-cement ratio could not be met using the normal quantity of materials specified.

To acquaint engineers and inspectors on each contract with the effects of variations in materials on concrete, trial batches were made in the field before the start of paving operations wherever possible. These trial batches served also as indicators of water requirements to

be expected and furnished information for establishing a starting mix. Final mix adjustments were always made in the field in conjunction with careful observation of concrete placing operations. In general, the first concrete placed on each contract contained somewhat more sand than was expected to be necessary. Changes were made after a sufficient quantity of concrete had been placed to establish characteristics of the mix under trial, but such changes were made only under the supervision of a qualified engineer. Proportions of the three aggregate sizes varied considerably on different contracts, probably due largely to variations in physical properties of the aggregates themselves and also, though to a lesser extent, in characteristics of cement and equipment used in placing and finishing. The proportion of material finer than 4-mesh to the total aggregate (approximately the proportion of sand to total aggregate) varied between 34 to 37 per cent by absolute volume. Some mixes were highly sensitive to sand content, as indicated by behavior during transverse screeding and other finishing operations, while others permitted an appreciable change in sand content without noticeable change in workability or finishing characteristics. No attempt was made to reduce mixing water by using harsh mixes of difficult placing properties. Variations in proportions of the two sizes of coarse aggregates varied between 45 to 55 per cent and 55 to 45 per cent, respectively, depending on their effect on the characteristics of the concrete.

CONTROL PROCEDURE

Control of water-cement ratio depended on establishing and maintaining a relation between water-cement ratio and consistency as measured by concrete slump. The procedure adopted required uniformity in physical properties of materials and close and intelligent attention to details by the inspectors in charge of the work. It required cooperation between the inspector at the batching plant and the inspector at the paver, and an earnest desire on the part of both to provide the accurate data which were essential to proper functioning of the control procedure. Since all Turnpike paving concrete in the 29 paving contracts noted was mixed and placed by boom and bucket pavers, and all materials were batched in weight batching plants, the control procedure was the same in each case.

Moisture determinations on each size of aggregate were made in a small control laboratory at each batching plant by oven-drying representative samples of each material. Total moistures thus found were corrected for absorption by amounts previously determined

by repeated laboratory tests on the materials and were used in computing the amount of surface water added to each batch of concrete with the aggregates. Aggregate batch weights were corrected for the amount of contained moisture using the results obtained by the moisture determination. Information thus obtained was transmitted to the mixer inspector on especially prepared batcher-mixer slips. The records on these slips were completed by entering the quantity of batched mixing water, the slump measurement, calculation of water-cement ratio, and such remarks as appeared pertinent. A notice carried by a batch truck driver advised the mixer inspector of the materials batch upon which moisture determinations were being made and allowed him to obtain his data on concrete containing those same materials for entry on the batcher mixer slip when it arrived somewhat later.

In using this information, no attempt was made to control mixing water on an individual set of moisture tests. There were two reasons for this. In the first place, the moisture determinations lagged 15-20 minutes behind concrete placing even under the best of conditions. Second, individual determinations, even if available immediately, were not considered of sufficient reliability to permit control directly on a water-cement ratio basis. However, a reasonable number of consecutive determinations of mixing water and consistency permitted the latter to serve as a measure for both. Slump then, continuously checked by mixing water determinations, furnished the working basis for control of water-cement ratio.

Tests on moisture content of aggregates at the batching plant and again at the paver indicated that for very long hauls during dry, windy weather, a loss in moisture of at least a gallon from the damp materials required for a 37-cu. ft. batch of concrete could be expected under some conditions. No attempt was made to correct water-cement ratios by this loss, however, because of rapid variations in conditions affecting such loss of moisture. The error introduced was on the side of safety since water ratios were slightly lower than the calculations indicated while the foregoing conditions existed.

As noted above, successful operation of the control procedure depended on uniformity of materials. Because of the large quantity of aggregate required in a short time, stockpiling was necessary, particularly in the case of sand which had to be transported over long distances. Because of this, the 12-hour stockpiling of sand required by the specification seldom became an issue, and reasonable uniformity in moisture content was general. Coarse aggregates were stockpiled on all contracts but to a much lesser extent, and

with no requirements for stockpiling, occasionally caused control problems due to variation in moisture content. Stockpiling and reclaiming operations were closely controlled to minimize segregation. Routine screen analyses, and wash tests when required, supplied information on grading and served as acceptance tests for the aggregates as received for stockpiling and as reclaimed for use in concrete.

The importance of obtaining truly accurate records was emphasized at every opportunity, and it is believed that the records, in general, are representative of the work.

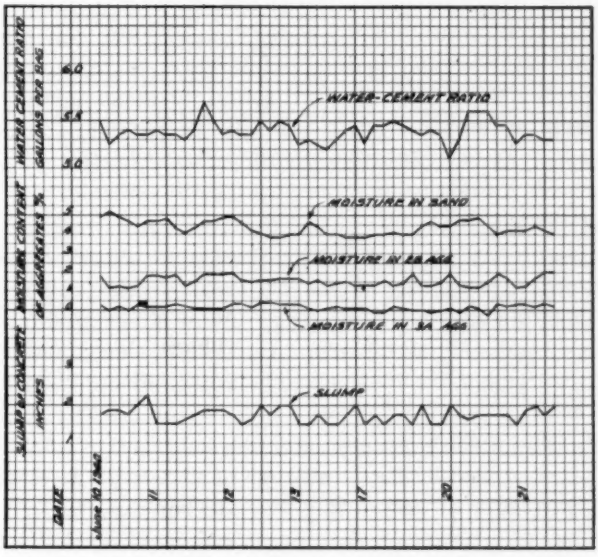

FIG. 2—MOISTURE-CONSISTENCY CONTROL DATA
Water-cement ratio values calculated

The chart shown in Fig. 2 is typical of slump-moisture control records obtained on each contract. The water-cement ratios shown were calculated from moisture determinations and water batcher settings. Slumps indicated were measured on concrete containing materials on which moisture determinations had been made. It will be noted that correlation of individual determinations is not good, probably mainly because of errors in sampling, but it is believed that actual water-cement ratios are well within the limits of the high and low points shown.

It is of particular interest to note that the average water-cement ratio for all Turnpike pavement concrete, in which crushed stone was

used as coarse aggregate, was about 5.4 gal. per sack with slump averaging 2 inches or slightly more. This is considerably higher than some paving records have shown for similar mixes, and leads to the suspicion that perhaps considerably more mixing water may be used in paving concrete than is generally supposed. It seems safe to state that it would have been impossible to obtain concrete of a consistent 3-in. slump with 6¼ sacks of cement per cubic yard and a water ratio not exceeding 5½ gal. per sack on the average Turnpike contract.

Another fact brought out conclusively was that different materials, each meeting specification requirements, caused appreciable differences in the amount of mixing water required for concretes having equal cement content and slump. Differences of as much as ½ gal. of water per sack of cement were found in some instances. It is believed that the greatest effect was due to particle shape and surface texture of the sand, although the characteristics of the coarse aggregates were known to have an appreciable effect.

Every effort was made to control concrete consistency on the basis of its compactibility under the transverse screed rather than on the basis of finishing qualities of the concrete surface or the time lag between screeding and final finishing. After a few preliminary difficulties, the usual eleven o'clock (A. M.) finishing nightmare was fairly well controlled without using a half gallon of "extra" mixing water per sack of cement. This was accomplished by giving closer attention, both by the inspector and the contractor, to the finishing procedure during that critical interval of the day's operations, and consisted essentially of scheduling the finisher's shifts so as to provide sufficient man-power at the time it was most needed. Inasmuch as finishing ability of concrete pavement, from the viewpoint of the average finisher, is dependent to a considerable extent on bleeding, the natural tendency was to use more mixing water for concrete containing materials of low bleeding tendencies than for concrete which bled freely even at stiff consistencies. By slightly revising the finishing procedures and maintaining an adequate force of skilled finishers, no insurmountable finishing difficulties were encountered with Turnpike concrete having slumps of 1½ to 2 inches, regardless of bleeding characteristics of the concrete.

By using the stiffest consistency of concrete which was readily placeable, and mix proportions of only slightly "oversanded" characteristics, a thin surface layer of mortar could be maintained. Excessive oversanding, or slightly increasing the mixing water above the minimum required for workability increased the thickness of

the mortar layer very rapidly. Finishing operations subsequent to transverse screeding were designed to further reduce thickness of the mortar layer by removing surface accumulations. Too much emphasis cannot be directed to the importance of removing watery accumulations on the pavement surface rather than incorporating them with the concrete below.

Preliminary strength tests on trial mix concrete and routine tests on concrete mixed for the work were made. Six by 8 in. beams and 6 x 12 in. cylinders were prepared. In general beams indicated breaking strengths of about 700 p.s.i. at 5 days. A relatively small number of the 6 in. cores drilled for pavement depth determinations which were tested for compressive strength indicated 5,000 to 6,000 p.s.i. at ages of a month or more. Unfortunately information is not available for preparation of a detailed report on test results.

PAVING EQUIPMENT AND PROCEDURE

Prevailing weather was cool and damp during much of the spring paving period with many rainy days, which required suspension of construction operations entirely. These conditions were unfavorable to rapid construction, actually causing a considerable delay in the completion date, but were definitely favorable to the placing of high quality pavement. Except during the comparatively few hot, dry windy days encountered, many of the difficulties associated with the placement of paving concrete during hot weather were avoided. The exceptions served to emphasize the effect of atmospheric conditions on paving operations generally. Successful night operations on several of the contracts suggested many advantages of night over daytime paving construction during hot seasons.

The typical paving contract used a 34E double drum mixer, one spreader, one or two transverse finishing machines, and a longitudinal finisher as major power equipment for mixing and placing concrete. It operated on two seven-hour shifts. Departure from this layout of power equipment varied considerable, depending on size of contract and the time available for its completion. A few contracts used 27E single drum pavers only and several used two pavers in tandem, a double drum for placing concrete below the reinforcing steel and a single drum to complete the lift after the steel was placed. The slower operations had a distinct advantage in that only a comparatively small area of incompleted pavement surface remained exposed with the result that finishing and protection problems during rapid atmospheric changes were seldom of a serious nature. Tandem operation of pavers during early stages of construction were not

entirely satisfactory due to a tendency toward keeping the pavers too far apart. Later operations placed the pavers boom to boom and allowed very little time to elapse between placement of the two layers of concrete. All pavers, it should be mentioned, operated at the side of the lane under construction. Several methods of limiting the distance between tandem pavers were tried, the most effective observed by the writer being to connect the pavers by a common water supply hose of just the length which would permit the maximum desirable distance between. The tandem paver operations which sometimes exceeded a paving rate of 300 lineal feet per hour in placing a single 12-foot lane, required very close coordination between all phases of the construction. With perfect organizations, very efficient tandem operation was possible, but under normal conditions when paving schedules permitted, it appears likely that the use of one paver was the more satisfactory.

Efficient operation of any of the equipment combinations depended on spreading and transverse screeding with the least possible delay after concrete was deposited on the subgrade. Changes in workability of concrete, which was placed with a reasonably stiff consistency, were rapid, particularly with higher placing temperatures and drying atmospheric conditions. Delays of 5 to 10 minutes were sometimes of serious consequences due to initial stiffening during this interval. The most successful contractors developed procedures which permitted spreading and transverse screeding immediately after the concrete was deposited.

Mechanical spreading of pavement concrete was used extensively on the Turnpike, where, in the writer's opinion, its advantages were adequately demonstrated. For proper operation the mechanical spreaders required a full load of concrete, particularly for the first strike-off below the reinforcing steel. For best operation a full load was also required for the second strike-off. Some fears expressed early during construction that segregation and other undesirable effects due to the mechanical spreading might be anticipated could not be verified from an inspection of more than a thousand 6-in. cores which had been drilled for measuring pavement thickness. Avoidance of delays and promotion of uniform rate of concrete placement are believed to well justify the use of this type of equipment.

The advantage of using extremely powerful transverse finishing machines, capable of carrying heavy loads of concrete ahead of the first screed during the first pass of the machine, could not be doubted by anyone who observed critically the operations on Turnpike paving. On one contract, when outmoded transverse finishers were used prior

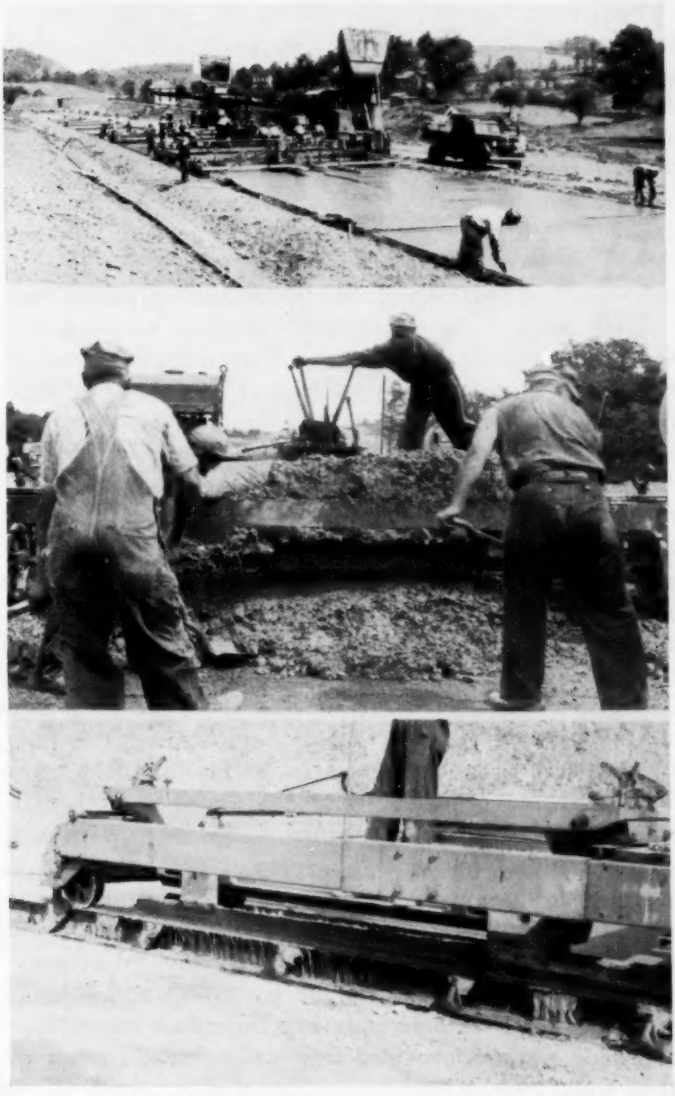

FIG. 3 (TOP)—TANDEM PAVER OPERATION

FIG. 4 (CENTER)—TYPICAL PAVING CONCRETE—APPROXIMATELY 1½-IN. SLUMP

FIG. 5 (BOTTOM)—LONGITUDINAL FINISHER COMPLETING PASS TOWARD CAMERA

to delivery of modern equipment, a change in production rate of about 50 per cent was made possible by the new equipment, which placed concrete of appreciably lower water content at approximately full capacity rate. It was estimated that concrete of at least 4-in. slump would have been required in order to allow full capacity production with the old machines on this contract. This, of course, was not permitted.

Longitudinal finishers also made their appearance in force on the Turnpike construction, giving an excellent account of themselves when close supervision was maintained. Like other equipment, their operation had to be closely controlled and correlated with other phases of the work. For satisfactory results it was necessary that screeds of transverse machines be so adjusted that the concrete surface was slightly above finished grade after settlement at which time the longitudinal screeding was done. Complete discharge over the forms at the end of each longitudinal screed pass was required in order to remove surface water and accumulations and to avoid remixing such material with surface mortar. The amount of hand finishing was materially reduced by the use of this machine.

Hand surface finishing was kept to a minimum consistent with accuracy of surface contour required, but the demand for excellence in riding qualities necessitated some surface manipulation following the power finishing equipment. For this work, the use of scraping straight edges, required for some time by the Pennsylvania Department of Highways, rather than smoothing floats, was encouraged because by their use excess water and sloppy mortar could be removed from the pavement surface. Floats having large areas in contact with the surface did little more than massage the surface and remix surface water with the mortar.

Tank trucks for supplying mixing and curing water were used on many contracts, and several advantages over the usual pump and pipeline were apparent. Mixing water was provided at appreciably lower temperature than was possible by pump and long pipeline delivery during hot weather, and thus improved concreting conditions. Sprinkling trucks provided more uniform application of curing water than could be obtained by sprinkling with a hose. It appears likely that these advantages would have been even more apparent had the paving schedule extended another month into the summer. Advantages of tank delivery were more pronounced on the longer contracts.

Curing of pavement received somewhat more than ordinary attention on Turnpike construction. Wet curing beginning as soon as the

FIG. 6 (TOP)—SUCH SURFACE ACCUMULATIONS WASTED (IN GENERAL)

FIG. 7 (BOTTOM)—HAND FINISHING

fresh concrete surface would permit was required for the first 24 hours after which several alternatives were permitted, the most popular combination being the required 24-hour wet period followed by paper curing for 48 hours. Thorough wetting of pavement before covering with paper was required, and in all cases observed by the writer, moisture remained under the paper at the end of the curing period. Wet curing was accomplished by means of pipeline and sprinkling

hose or by sprinkling trucks, the latter equipment generally providing the more satisfactory results.

CONCLUSIONS

Paving construction on the Pennsylvania Turnpike offered an excellent opportunity to observe the operation of paving equipment, contractor's procedure, and engineering control on many contracts in simultaneous operation. Effects of materials, procedures, and equipment on characteristics of pavement concrete could be studied to advantage. Because of capable engineering administration of the project and excellent cooperation of contractors and materials producers, conditions were favorable for the production of high quality concrete in spite of the pressure for completion under which the work was carried out. The following observations relative to this work are the result of a study of various phases of the paving construction.

(1) Field control of mixing water and other factors affecting concrete quality is by no means easily accomplished on any paving projects, and it is not contended that anything like perfection was reached on Turnpike construction. However, it is believed that the results obtained were well worth the efforts made in the direction of closer control.

(2) It is possible to maintain uniformity of water-cement ratio and consistency of concrete within reasonable limits by application of well-known methods, provided the inspection personnel is of high cailber and is encouraged and supported in its actions by the engineer in charge.

(3) It is entirely practical to control concrete consistency on a basis of placeability under action of the transverse screed without giving consideration to hand finishing characteristics. A strong tendency to adjust consistency so as to provide a concrete most suitable to final hand manipulation of the surface required constant attention and some changes in finishing procedure for correction.

(4) Final adjustment of concrete mix proportions should be made in the field under closely observed full scale operating conditions. On Turnpike paving the average mix contained approximately 35 per cent sand and 65 per cent coarse aggregate by absolute volume (the 4-mesh screen being the dividing point) with about 34 and 37 per cent representing the limits of variation in sand content.

(5) Construction technique has a marked effect on placeability of paving concrete. It was noted that the most efficient operations were those on which the least possible time elapsed between deposition of concrete on the subgrade and transverse screeding of the slab.

(6) Type of equipment, particularly the transverse finishing machine, has a marked effect on consistency (and water content) of concrete which can be satisfactorily consolidated. Obsolete, or poorly designed equipment should be eliminated from important work by specification requirement.

(7) Finishing operations require close attention. Removal and wasting of surface material and bleeding water accumulated by finishing operations which take place after the first pass of the transverse finishing machine were considered essential for first quality pavement.

(8) Characteristics of materials have a marked effect on water requirements of the concrete in which they are used. Differences approaching ½ gal. of water per sack of cement were indicated for concrete of Turnpike mix proportions and consistency within the range of materials supplied for this project. Effect of characteristics of materials on bleeding of concrete was even more apparent, suggesting still greater variations in water-cement ratio, had finishing characteristics been allowed to influence the mixing water content.

* * * * * * *

The writer takes this opportunity to express his appreciation to Samuel W. Marshall, Chief Engineer of the Pennsylvania Turnpike Commission and members of the design and construction staff through whose efforts the control measures described were put into effect. In this connection, it should in fairness be stated that paving contractors showed an almost equal interest and cooperation.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the* JOURNAL *Supplement concluding this Proceedings volume

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JOURNAL
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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by I. L. Tyler:

Concrete Control on the Pennsylvania Turnpike*

BY MR. TYLER, H. HERSHEY MILLER AND HAROLD ALLEN

CONVENTION DISCUSSION

Mr. Tyler—We could not find more than two out of the 29 contracts that required exactly the same proportions of material.

There have been some stories about the Turnpike that are slight exaggerations. I believe you hear about half inch slump. I think this was exceptional rather than the rule. We did not have equipment which could handle concrete of that slump.

Consistency was governed by placeability under the transverse finishing machine and not by later finishing operations. There was a tendency to try to regulate water to suit later finishing conditions, but to the best of our ability we discouraged that tendency.

It was seldom that we were able to make a trial mix of concrete and wind up with that mix on the job. Placing the different types of equipment required appreciable differences in proportioning to take care of the conditions. In one instance two different jobs were batched from the same plant, both had exactly the same proportions, and I believe any impartial observer would have decided that one was over sanded and the other was under sanded. We compromised in that case.

I believe that any one would find it difficult to describe the method of setting the screed for the best operation of the longitudinal finisher. This was a matter of trial and error until we got the right elevation. Sometimes there were some difficulties attached to high super elevations on the steeper grades.

*A. C. I. JOURNAL, Feb. 1941; *Proceedings* V. 37, p. 361.

According to rumor one of our construction engineers, after he had made a trip over his district, sent a request to each contractor to allow only one float on the job. On his next inspection he found about the same number as the first time, so he broke up all but one float for each finishing operation.

*H. Hershey Miller**—I do not have a written discussion of this paper, but I was associated with Mr. Tyler on the Turnpike and I think that he has been extremely modest in his presentation of this subject.

On a number of these projects we had two and three shifts running, which meant that it was necessary to break in a lot of men as concrete men and as plant inspectors who had no previous knowledge of this work. The project was roughly divided into four districts, and each one of these districts was in charge of a concrete technician whose business it was to instruct the inspector on the individual project and see that the specifications were carried out. As brought out by Mr. Tyler, speed was one of the essential factors and naturally the contractors and even our own personnel hid behind this speed factor in trying to evade some specification requirements. Much pressure was brought to bear upon the materials division and I must say that Mr. Tyler stood adamant against any deviations. The specifications as used on the Pennsylvania Turnpike follow modern good practice, and other pavements built under similar specifications have lasted *at least* ten years. So, with the control that was exercised on the Turnpike, I feel sure that the concrete in the Turnpike pavement will last well over the ten year period and I hope to be alive for a good many years so that I can see this concrete go to pieces. One of the things brought out by Mr. Tyler in his paper which he did not touch on in his presentation here was the concrete tests, and one of the interesting things that I jotted down from some data on modulus was the difference in the strength of the concrete from the eastern and the western sections of the Turnpike. The Allegheny Mountains put the eastern section of the Turnpike south and east of the mountains, and it is interesting to note that while the average strength of concrete at five days east of the mountains was 736 lb.; west of the Alleghenies where more unseasonable weather conditions existed, the average was 655 lb. At the seven day period, the concrete in the east was 835 lb. and in the western section it was 702 lb. You must remember that while these strengths were not exceptional, this pavement was placed October to November, part in December, March, April, May, June

*Pennsylvania Sand and Gravel Producers Association.

and July and the strengths in July are well above the averages mentioned.

BY HAROLD ALLEN*

The paper which Mr. Tyler has presented demonstrates very clearly that it is practical to exercise accurate and adequate field control in the mass production of concrete.

The experience of the writer upon which this discussion is based began in 1930 and was continued through 1937. During those years a water-cement ratio specification was used in the construction of all concrete pavements and structures placed on the Kansas highway system.

In 1929 it became apparent to Kansas highway engineers that the use of arbitrary proportions and fixed cement factors were neither economical nor desirable. Also, research carried on in the Kansas State College laboratories indicated that the durability of concrete composed of sound aggregates was directly related to the water content. Therefore, the water-cement ratio as the basis of all concrete specifications was adopted in 1930 and has been used continuously since that time.

On trial projects, in which the water-cement ratio was made the controlling factor, it was found that if a specification of this type was to be used successfully, careful attention would have to be given to (1) personnel, (2) gradation of aggregates, (3) handling of aggregates, (4) control of moisture in aggregates.

Throughout the seven years of this writer's experience, rigid control of these factors in the application of the water-cement ratio method resulted in concrete which was durable and had adequate strength.

Control by the water-cement ratio method requires a well informed personnel of such size that adequate attention can be given to all details.

It has been my experience that young engineering graduates, who are not afraid of work, trained for a month to six weeks or longer in the laboratory, make the best proportioning plant inspectors. An experienced chief inspector or field engineer should be assigned to work with each new inspector until he becomes familiar with his job and to supervise the work of inspectors on all projects on which concrete is used. The area which such a field engineer is required to cover should be small enough that he can reach any project under his supervision within a few hours. Field engineers and inspectors must be enthusiastic about their jobs and must be willing to give close attention to details and complete cooperation to their associates.

*Materials Engineer, Division of Tests, Public Roads Administration, Washington.

The administrative officer responsible for the fabrication of large volumes of concrete must spend some of his time on each of the projects under constructlon if he is to cope with the difficulties encountered and the temptations for deviation from specifications which the proportioning plant engineers must overcome.

If the water-cement ratio specification is to be used successfully, all aggregates must be graded within narrow limits and must be stock piled and handled in such a manner that they will not be segregated.

Special provisions in coarse aggregate specifications are not usually necessary in order to insure uniformity. A narrow range in gradation is usually specified and the preparation of the material is usually controlled adequately at the crushing plant.

Special attention must be given to fine aggregates because fluctuations in gradation will effect the water requirements for the mix. A clause in the specifications limiting the maximum variation in the fineness modulus to plus or minus 0.15 from a predetermined average for aggregate from a given source is effective in insuring suitable uniformity. For example, if a plant chooses to supply material having a fineness modulus of 2.50, all of the fine aggregate from this source must have a fineness modulus of not less than 2.35 or more than 2.65 in addition to meeting all other requirements. Uniformity may also be obtained by specifying narrow limits for the percentage retained on each sieve size. The following gradation requirements were found to be satisfactory for fine aggregates available in Kansas.

Sieve Size	Percent Passing
⅜-inch	100
No. 4	95-100
No. 16	65- 80
No. 50	5- 10
No. 100	0- 2

In this specification, the range of the percents passing each sieve is much narrower than the requirements shown by Mr. Tyler. Also, the requirements for the percentages passing the No. 50 and No. 100 sieves do not conform to the requirements recommended in the paper. These fractions are low because Kansas sands in the natural state contain only small amounts of fine material. With the small quantity of fines available, concretes of suitable workability for placing were produced and after 10 years of service have shown satisfactory durability.

The stock piling of coarse aggregates is not a difficult problem when the use of several separate sizes is specified. When one aggregate is used, segregation may be prevented by placing the stone in layers,

not more than 4 ft. thick with a berm not less than 5 ft. wide at the top of each lift thus preventing the coarse particles from rolling to the bottom of the pile.

The proper handling of aggregates from the stock piles to the bins is essential to close control. The proportioning plant engineer should direct the operations of loading equipment so that the aggregates passing through the weighing hoppers will have about average gradations and moisture contents.

The stock piling of wet aggregates must be done far enough in advance of concreting operations to insure a constant moisture content in the aggregate. When the proportioning plant is set up adjacent to the aggregate producing plant or when the haul is very short, it may be necessary to stock pile aggregates from three to five days before the moisture content becomes uniform. Specifications which provide that aggregates be stock piled until successive tests indicate a constant moisture content are more suitable for adequate control than those in which a definite length of time is specified. Variable drainage characteristics of aggregates make the choice of a fixed period for stock piling difficult.

The method for the determination of the moisture content of aggregates during concreting operations should be accurate and fast. The displacement method developed by Professor Dunagan of Iowa State College has been in use in Kansas since 1930. It is rapid and reliable. Moisture tests can be made at five minute intervals if necessary and no lag between the proportioning plant and mixer in the control of the quantity of mixing water need occur.

Properly proportioned and mixed concrete cannot be placed and finished with equipment which is inadequate or poorly adjusted. The over sanding of concrete mixtures and the use of excessive water are often necessary because the equipment does not operate properly. The economy in the use of the water-cement ratio and the value of adequate proportioning plant and mixer control may be lost unless the paving engineer gets the maximum efficiency from properly designed finishing equipment.

In conclusion, the observation of Mr. Tyler regarding the importance of obtaining truly accurate records on each project cannot be over-emphasized. Every concrete structure or pavement may be regarded as a research project and complete records will be very valuable for future study.

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Proposed
Specifications for Concrete Pavements and Bases*

By American Concrete Institute Committee 617, Specifications and Recommended Practice for Concrete Pavement and Base†

H. F. CLEMMER, CHAIRMAN

Seven of the eight members of Committee 617 approved the report as here submitted to the Institute (one member not participating in committee's work). The Standards Committee has released it for publication for a period of general study and discussion but not at this time for formal presentation to the Institute on motion for adoption as an A. C. I. standard. Such release would presumably depend on the reactions to the report in the discussion period.

SCOPE

These specifications apply to pavements and pavement bases constructed of portland cement concrete, with or without reinforcement as specified, constructed on the prepared subgrade or other base course as indicated by these specifications and in conformity with the lines, grades, thicknesses and cross sections as specified or shown on the plans. Throughout these specifications certain minimum requirements are given, which are known to produce satisfactory results for average conditions. For other conditions of traffic and for unusual conditions of sub-grade, drainage, and exposure it will be necessary for the Engineer to make special provisions for subgrade treatment, additional strength in the pavement slab, and other pertinent factors.

*Submitted to the Institute Sept. 20, 1940; referred to Standards Committee, Sept. 23, 1940; in its present form released by Standards Committee Dec. 28, 1940, for A. C. I. publication and discussion.

†Committee 617 has the following membership: H. F. Clemmer, Chairman, R. D. Bradbury, A. T. Goldbeck, Fred Hubbard, F. H. Jackson, Bailey Tremper, Stanton Walker, W. H. Wood. Seven of the eight members voted affirmatively for the report as here presented, none negatively.

PART I—MATERIALS

CONCRETE

Constituent Materials

Approval of Sources of Material Supply

Sources of supply for portland cement, aggregates and water shall be approved by the Engineer before the shipment of materials is started. Aggregates shall be furnished only from approved plants equipped with facilities adequate for the production of the material as specified and in such quantities as shall be required for the prompt execution of the contract.

Quality of Portland Cement

Cement shall conform to the requirements of the current Specifications for Portland Cement of the A.S.T.M. (Designation: C 150) or to the requirements of the current Standard Specifications for High Early Strength Portland Cement of the A.S.T.M. (Designation: C 74) as may be required. All tests shall be made in accordance with the current Standard Methods of Sampling and Testing Portland Cement of the A.S.T.M. (Designation: C 77).

Transportation and Handling of Cement

All cement shall be protected against loss during handling, in transit, and from the weather, under all conditions. Either package or bulk cement may be used. When package cement is used, the cement shall be emptied from the sacks into the batch immediately prior to mixing. When bulk cement is used, satisfactory methods of handling and weighing must be employed. A separate hopper shall always be used for weighing cement. The cement handling equipment shall be so designed and operated that the quantity of cement for each batch shall be maintained in a separate container so as not to come in direct contact with the aggregates.

Quality of Aggregates

Aggregates shall conform to the requirements of the current Standard Specifications for Concrete Aggregates of the A. S. T. M. (Designation: C33). The coarse aggregates shall be shipped in at least two separate sizes and combined at the proportioning plant in the relative amounts fixed by the Engineer. The two sizes of coarse aggregates shall be separated on the ¾ in. sieve when the grading requirements are from No. 4 to 1½ in., and on the 1 in. sieve when the grading requirements are from No. 4 to 2 in.

Storage and Protection of Aggregates

Aggregates of different kinds or sizes shall be placed in different stock piles. Aggregates shall not be placed or stored on the finished subgrade and positive means shall be used to prevent the inclusion of foreign materials. Frozen aggregates or aggregates containing lumps of frozen material shall be thawed before use. Aggregates shall be transported to the mixer in such a manner as to insure that the required amount of each size of aggregate is obtained in each batch of concrete.

Stock Piling of Aggregates

The formation of conical stock piles of coarse aggregate by the deposition of materials in one place will not be permitted. Stock piles of coarse aggregates shall be built up in successive horizontal layers, of which each layer shall not be more than three feet thick. Each layer shall be completely in place before the next is started. Should the aggregate become segregated, it shall be remixed so as to conform to the grading requirements of the specifications. All aggregates produced or manipulated by hydraulic methods and all washed aggregates shall be handled in such a manner as to permit drainage for at least 12 hours before use.

Water

All water used in mixing or curing concrete shall be clean, and free from oil, salt or acid, vegetable or other substance injurious to the finished product. Where the sources of water are relatively shallow, they shall be maintained at such a depth and the intake so enclosed as to exclude silt, mud, grass or other foreign materials.

Proportioning

The Engineer shall specify either of the following methods of proportioning: Proportions Based on Design for Minimum Strength, or Proportions Based on Uniform Cement Factor.

Proportions Based on Design for Minimum Strength

(*a*) The proportions of cement, fine aggregate, coarse aggregate and water to be used in the mix shall be fixed by the Engineer on the basis of laboratory tests of concrete in which are used aggregates from the same sources and of the same gradings as will be employed in the work, and using that portland cement which is found to produce the lowest strength concrete of any acceptable cement available for the work.

(*b*) The mixture shall be designed to produce workable concrete having a slump of two inches to three inches for unvibrated concrete or one inch to one and one-half inch for vibrated concrete, and a modulus of rupture at fourteen days of not less than five hundred fifty (550) p.s.i., as determined from specimens made, cured and tested in the laboratory in accordance with the current Standard Method of Test for Flexural Strength of Concrete, (Laboratory Method Using Simple Beam with Third Point Loading) of the A. S. T. M. (Designation: C78).

(*c*) The mixing water, including free surface moisture on the aggregates but exclusive of moisture absorbed by the aggregates, shall not exceed six gallons per sack of cement in localities where the concrete will be subjected to severe freezing and thawing conditions; and in no case shall the mixing water exceed six and one-half gallons per sack of cement nor shall the cement content be less than five sacks per cubic yard.

Note—In the practical application of a specification of this type it has been found that certain additional requirements are necessary in order to define accurately the duties and responsibilities of the parties of the contract. The following additional requirements are suggested:

(1) The Engineer will furnish prospective bidders, upon request, the proportions by weight which will be required for all aggregates from established commercial sources available for use on the project. This information will also include the grading of the aggregates used in determining these properties.

(2) Immediately upon receipt of notice of award of the contract, the contractor shall furnish the Engineer with the exact location or locations of the source or sources of aggregates which he proposes to use. The proportions will then be designated by the Engineer. Such proportions will govern as long as materials are furnished from the sources originally designated and so long as they continue to meet the requirements specified, subject only to slight variations in the relative quantities of fine and coarse aggregates for the purpose of securing adequate workability.

(3) If, during the progress of the work, the contractor wishes to use aggregates from sources other than those originally designated by him, the Engineer reserves the right to change the proportions in order to meet the minimum requirements for modulus of rupture hereinbefore specified with the aggregates actually furnished.

(4) In case satisfactory plasticity and workability are not secured using the proportions and aggregates originally designated, the Engineer may alter such proportions. If such alterations change the designated cement factor or factors originally fixed by two per cent or less, no adjustment in the amount paid the contractor will be made. If such alterations change the designated cement factor by more than two per cent, the source and quality of the aggregates remaining the same, payment will be adjusted for or against the contractor in whatever amount the total cost of materials, f.o.b. contractor's material yard, has been increased or decreased by more than two per cent. The amount of such increase or decrease shall be calculated from the designated cement factor and not by count of bags of cement used or of the batches where bulk cement is used.

Proportions Based on Uniform Cement Factor

This method is based on arbitrary proportions which have been shown by experience to give satisfactory results and designed to produce a constant yield, regardless of the source or kind of materials. A table of factors to be used in determining weights of saturated surface-dry aggregates per bag of cement for a mixture having a cement factor of six bags per cubic yard is included. These factors when multiplied by the respective bulk specific gravities will give the required quantities of fine and coarse aggregate per bag of cement. Alternate proportions, depending upon the method of compacting and finishing the concrete, are provided.

(*a*) Except as provided herein the following proportions of saturated surface-dry aggregates per bag of cement shall govern, and the batch weights of the materials used in the concrete mixture shall be set by the Engineer accordingly.

FACTORS USED FOR DETERMINING WEIGHTS OF SATURATED SURFACE-DRY FINE AND COARSE AGGREGATE PER BAG (94 LB.) OF CEMENT

Mixture	Cement Bags per Cubic Yard	Maximum Net Water Content per Bag of Cement Gal.	Slump in Inches	Factor x bulk specific gravity = lbs. of saturated surface-dry aggregate per bag of cement								
				When gravel is used as coarse aggregate			When crushed stone is used as coarse aggregate			When slag is used as coarse aggregate		
					Gravel			Crushed Stone			Slag	
				Sand	Fine Size	Coarse Size	Sand	Fine Size	Coarse Size	Sand	Fine Size	Coarse Size
A	6.0	6.0	2–3	66	53	81	75	49	76	79	48	73
B	5.5	6.0	1–1½	66	64	96	75	60	91	79	59	88
C	6.0	5.3	1–1½	60	59	87	68	55	83	71	54	81

Note: Using the maximum allowable water content, the weights as determined by the above factors (when multiplied by the appropriate number of bags of cement per cubic yard) will produce a cubic yard of concrete (absolute volume). (If an amount of water less than the maximum is used the weights of aggregates shall be adjusted to maintain the same minimum cement factor as indicated.)

(*b*) Mixture (A) shall be used where the concrete is to be compacted and finished with a standard finishing machine without vibration. Mixture (B) shall be used where the concrete is to be compacted and finished by vibration and it is desired to maintain the same water-cement ratio as when the standard finishing machine without vibration is used. Mixture (C) shall be used where the concrete is to be compacted and finished by vibration and where it is desired to maintain the same cement content as when the standard finishing machine is used. The Engineer shall designate the mixture to be used on each specific project.

(*c*) Since the weights as determined from the table are computed for aggregates in a saturated surface-dry condition, the batch weights shall be corrected to accord with the moisture condition of the aggregates delivered to the measuring bin.

(*d*) The weights of aggregates as determined from the table have been calculated on the basis of using a well-graded natural sand in combination with a coarse aggregate graded either from No. 4 to ¾ in. and from ¾ in. to 1½ in., or with a coarse aggregate graded from No. 4 to 1 in. and from 1 in. to 2 in. The ratio of fine to coarse aggregate may be changed, depending upon the gradation of the materials furnished by the contractor. The weights of fine and coarse aggregate will be adjusted by the Engineer to insure concrete of satisfactory plasticity and workability, using not more than the maximum permissible net water content shown. During the progress of the work, the total weight of aggregate per bag of cement, except as necessary to compensate for differences in specific gravity, shall not be changed except under the following conditions:

(*e*) If concrete of satisfactory plasticity and workability cannot be made without exceeding the maximum net water content, the Engineer shall reduce the total weight of aggregate as computed from the table by an amount sufficient to insure that the maximum net water content will not be exceeded, and the contractor shall not receive additional compensation for any extra cement which may be necessary by reason of such adjustment.

Measurement of Materials

Materials shall be measured by weighing, except as otherwise specified or where other methods are specifically authorized by the Engineer. The apparatus provided for weighing the aggregates and cement shall be suitably designed and constructed for this purpose. Each size of aggregate and the cement shall be weighed separately. The accuracy of all weighing devices shall be within one-half of one per cent throughout the range of use. Operation shall be required within a degree of accuracy of one per cent for cement and water, and two per cent for aggregates. Cement in standard packages (sacks) need not be weighed. The use of fractional packages will not be permitted unless weighed. The mixing water shall be measured by volume or by weight. The water measuring device shall be susceptible of control accurate to one-half of one percent. All measuring devices shall be subject to approval.

Mixing

Mixing at Site

The concrete shall be mixed in a batch mixer conforming to the requirements of the Standards Committee of the Mixer Manufacturers Bureau of the Associated General Contractors of America which will insure a uniform distribution of the material throughout the mass, so that the mixture is uniform in color and homogeneous. Each batch of concrete materials shall be mixed for not less than one minute after all materials, exclusive of the mixing water, are in the mixer drum. The batch shall be so charged into the mixer that some water shall enter in advance of cement and aggregate, and shall continue to flow for a period which may extend to the end of the first one-third of the specified mixing time. The mixer shall rotate at the rate recommended by its manufacturer. Concrete shall be mixed only in such quantities as are required for current use, and any which has set so that it cannot be properly placed shall not be used. Retempering of concrete which has partially set, by mixing with additional water, will not be permitted.

The mixer shall be equipped with a suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked and so constructed that the water can be started only while the mixer is being charged. It shall also be equipped with an attachment for automatically locking the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged after each charge has been mixed the required time. A suitable means for discharging and spreading the concrete on the subgrade shall be provided. The mixer shall be cleaned at suitable intervals while in use. The volume of the mixed material per batch shall not exceed the manufacturer's normal rated capacity of the mixer, exclusive of the overload. The manufacturer shall install a plate upon the mixer stating the rated capacity and the recommended revolutions per minute.

Central Mixing Plant

The central mixer drum shall be of adequate size to accommodate the maximum batch, and shall be in accordance with the Concrete Mixer Standards adopted by the Mixer Manufacturers' Bureau of the Associated General Contractors of America. It shall be capable of combining the aggregates, cement, and water within specified time into a thoroughly mixed and uniform mass, and of discharging the

mixture without segregation. The mixer shall be rotated at the rate recommended by its manufacturer. Provision shall be made at the mixer to insure that the concrete is not discharged until the specified mixing time has elapsed. The mixing time shall be measured from the time that all materials, exclusive of mixing water, are in the mixer drum. When the central mixing plant is used for the complete mixing of concrete to be transported in an agitator truck, the minimum mixing time for mixers of one cubic yard capacity or less shall be not less than one minute; for mixers of larger capacities this mixing time shall be increased at the rate of fifteen seconds or more for each cubic yard, or fraction thereof, additional capacity. When the central mixing plant is used for the complete mixing of concrete to be transported in a non-agitating truck, the minimum mixing time for mixers of one cubic yard capacity or less shall not be less than one minute; for mixers of larger capacities this mixing time shall be increased at the rate of 30 seconds or more for each cubic yard, or fraction thereof, additional capacity.

Truck Mixers and Agitators

(1) *General* (*a*) The requirements for operating truck mixers and agitators covered herein are based on recommendations of the National Ready Mixed Concrete Association. These recommendations are based on equipment manufactured by companies holding certificates from the Association. The Association considers equipment only with regard to its ability to mix and/or transport concrete in an acceptable manner from the point of view of quality. The Standards Committee of the N. R. M. C. Assn. will study such data as manufacturers may present, and advise as to the data required, to determine under what limiting conditions as to capacity, mixing time, etc., their equipment should be operated, and will issue a certificate in that regard.

(*b*) These standards and recommended practices are subject to periodic revision as further studies justify such revision.

(2) *Definitions* (*a*) The two types of truck mixers and agitators covered herein are the following: (A) a closed or water-tight revolving drum suitably mounted and fitted with adequate blades attached to the drum; (B) an open-top water-tight trough-like container suitably mounted and fitted with adequate blades, revolving about an axis parallel to the axis of the trough.

(*b*) The truck mixers are capable of combining aggregates, cement and water into a thoroughly mixed and uniform mass of concrete and

of discharging the mixture with a minimum of segregation. The truck agitators are capable of transporting and discharging concrete with a minimum of segregation.

(*c*) Truck mixing is the process of combining the aggregates, cement and water into a thoroughly mixed and uniform mass of concrete, in a truck mixer.

(*d*) Shrink-mixing is the process of completing, while in transit to destination, the mixing of concrete which has been partially mixed in a stationary mixer the minimum amount of time required to incorporate the ingredients into a mass. (Note: About 30 seconds.)

(*e*) Agitating is the process of slowly mixing to prevent segregation in a truck mixer or a truck agitator while in transit to destination, concrete which has already been mixed in a stationary mixer.

(3) *Capacities* (*a*) When used for truck mixing, the capacity of truck mixers shall be in accordance with the manufacturer's rating but the volume of mixed concrete shall not exceed 57½ percent of the gross volume* of the drum or container.

(*b*) When used for shrink-mixing, the capacity of truck mixers shall be in accordance with the manufacturer's rating but the volume of mixed concrete shall not exceed 70 per cent of the gross volume* of the drum or container.

(*c*) When used for agitating, the capacity of the truck mixer or of the truck agitator shall be in accordance with the manufacturer's rating but the volume of mixed concrete shall not exceed 80 per cent of the gross volume* of the drum or container.

(*d*) The manufacturer shall attach to each truck mixer and truck agitator a metal plate on which is stated the capacities, in terms of volume of mixed concrete, for the various uses to which the equipment is applicable. When capacities less than the limits indicated herein are shown on the metal plate it indicates the maximum capacities for which the equipment was designed to be operated under the conditions stated by the manufacturer, and such capacities shall govern.

(4) *Mixing Speeds* (*a*) Mixing speed for the revolving drum type of truck mixer shall be not less than four revolutions per minute of the drum nor greater than a speed resulting in a peripheral velocity of the drum of two hundred twenty-five feet per minute. For the revolving blade type of mixer, mixing speeds shall not be less than six nor more than 16 revolutions per minute of the mixing blades.

*For the open-top mixer, the gross volume on which rated capacities are based shall be calculated on the basis that the vertical dimensions of the container above the central shaft shall not exceed the radius of the circular section below the central shaft.

(*b*) Agitating speed, for both the revolving drum and revolving blade types, shall be not less than two nor more than six revolutions per minute of the drum or of the mixing blades.

(5) *Mixing Time* (*a*) For truck mixers into which the *fine and coarse aggregates are charged simultaneously*, not less than 40 revolutions of the drum or blades at mixing speed shall be required after all of the ingredients including water are in the drum. (Note: Cement and water may be charged separately.)

(*b*) For truck mixers into which the *fine and coarse aggregates are charged separately*, not less than 60 revolutions of the drum or blades at mixing speed shall be required after all of the ingredients including the water are in the drum.

(*c*) For shrink-mixing, not less than 40 revolutions of the truck mixer drum or blades at mixing speeds shall be required.

(*d*) For truck mixing and shrink-mixing, not more than 150 revolutions of the drum or blades shall be at a speed in excess of that specified in Section 4 (*b*) as agitating speed. Additional mixing shall be at agitator speed.

(*e*) When means for verifying the mixing time is required, one of the following devices shall be employed:

(1) A batch meter and locking device mounted on the truck mixer capable of preventing the discharge of the mixer prior to the completion of the required number of revolutions.

(2) A counter mounted on the truck mixer permitting of reading the revolution count at the proportioning plant and at destination.

(6) *Time of Hauling.* Concrete shall be delivered to the site of the work, and discharged from the truck mixer or truck agitator completed within one and one-half hours (see note) after the cement comes in contact with the mixing water, or with aggregates which are surface wet.

(7) *Water Measurement* (*a*) For truck mixers, a water measuring device shall be provided to measure accurately the quantity of water for each batch. The device may be mounted on the truck mixer or located at the point of loading the truck mixer. It shall permit of ready access and of ready determination of the amount of water used.

(*b*) When wash water is used as a portion of the mixing water for succeeding batches, it shall be accurately measured and taken into account in determining the amount of additional mixing water required.

Note—Results of research have shown that one and one-half hours between the introduction of mixing water and the completion of the discharge of the batch results in no detrimental effects on the concrete. The data indicate that under specially

favorable conditions periods up to two and three hours may be allowed. When air temperatures are unusually high, or the ingredients of the concrete mixture are such that an unusually quick time of set or loss of plasticity may occur, it may be necessary to specify shorter periods to insure that no damage be done to the concrete; under these circumstances an investigation, satisfactory to the Engineer, with the materials to be used and in the proposed porportions, should be conducted to determine if a shorter period is necessary. It is unlikely that, under any circumstances, it will be necessary to substitute a period of less than 1 hour for the 1½ hours applicable to normal conditions.

Testing

Laboratory Flexure Tests

Flexural strength tests to be used as the basis for the design of concrete mixtures shall be made in accordance with the current Standard Method of Test for Flexural Strength of Concrete (Laboratory Method Using Simple Beam with Third Point Loading) of the A. S. T. M. (Designation: C78.)

Field Flexure Tests

When field flexural strength tests of the concrete are made for the project, the making and testing of the specimens shall conform to the following requirements:

If another method is used in making field tests than the simple beam with third point loading, as specified for laboratory tests, it is necessary to establish a ratio between the results of the two methods, in order to evaluate the results of the field tests.

(*a*) *Molding, Curing and Marking.* The concrete for the test specimens shall be taken from the concrete deposited on the subgrade and shall be carried to the molds. The material shall not be taken at random, but each shovelful shall represent as near as practicable the true proportions of the mortar and coarse aggregate as they occur in the mix, and shall be taken from a batch representative of the average consistency of the concrete being placed. In molding test specimens the concrete shall be placed in two equal layers. Each layer shall be rodded 50 times for each square foot of area. The top layer shall be such that the beam form is slightly heaped up with concrete. After rodding is completed for each layer, the specimen shall be spaded along the sides and ends with a mason's trowel or other suitable spading tool. When the rodding and spading operation is completed, the top shall be struck off with a straight edge and finished with a float.

The specimens shall be cured in as near as practicable the same manner as the pavement concrete hereinafter specified so that the

tests will directly represent the strength of the concrete in the pavement.

Specimens shall be properly identified as to date of molding and location of pavement represented. The weather conditions pertaining at the time of molding shall also be noted.

(*b*) *Number of Specimens.* At least two beams shall be made for each one thousand square yards of pavement surface.

(*c*) *Apparatus.* The following principles shall be secured in the design of the apparatus used for making flexure tests of concrete beams:

1. The distance between supports and points of load application shall remain constant for a given piece of apparatus.

2. The load shall be applied normal to the initial position of the neutral plane of the beam and in such a manner as to avoid eccentricity of loading.

3. The direction of the reactions shall, at all times throughout a given test, be parallel to the direction of the applied load. If this be vertical, then the direction of the reactions shall be vertical, and shall remain so throughout the test until rupture occurs.

Note—The more common devices, which, either singly or in combination, essentially secure such a relation between the applied loads and consequent reactions, include link supports, rocker bearings, spherical bearings or flexure plates.

4. The application of the load shall be such as to secure uniform increments of load for equal periods. This operation requirement demands that the operation of the apparatus be "smooth" and free from sudden shocks.

5. The ratio of distance between point of load application and the nearest reaction to depth of beam shall not be less than one.

Bulk Specific Gravity and Absorption of Aggregates

(*a*) *Fine Aggregate.* The specific gravity and absorption of fine aggregate shall be determined in accordance with the current Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate of the A. S. T. M. (Designation: C128).

(*b*) *Coarse Aggregate.* The specific gravity and absorption of coarse aggregate shall be determined in accordance with the current Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate of the A. S. T. M. (Designation: C127.)

Consistency

Consistency shall be determined in accordance with the current Standard Slump Test for Consistency of Concrete of the A. S. T. M. (Designation: C143.)

HIGH EARLY STRENGTH CONCRETE

All specifications for standard portland cement concrete shall apply to high early strength concrete. High early strength concrete shall

be produced by one of the following methods, or a combination thereof, as directed by the Engineer.

(*a*) By the use of high early strength portland cement in lieu of the standard portland cement specified and in the same amount.

(*b*) By the use of additional standard portland cement (the total amount of cement shall not exceed seven bags of cement per cubic yard of concrete).

(*c*) By the use of calcium chloride incorporated in the concrete mix in accordance with the current Standard Specifications for Curing Portland Cement Concrete Slabs with Calcium Chloride Admixture of the A. S. T. M. (Designation: C82.)

REINFORCEMENT STEEL

(1) *Steel Wire Fabric.* Steel wire fabric reinforcement shall conform to the requirements of the current Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement, of the A. S. T. M. (Designation: A185.)

(2) *Bar Mats.* The steel in bar mats shall conform with the current Standard Specifications for Bar or Rod Mats for Concrete Reinforcement, of the A. S. T. M. (Designation: A184.) Members shall be of the size and spacing shown on the plans. Unless otherwise shown on plans, members shall be spaced not more than 12 in. apart for longitudinal members and not more than 18 in. apart for transverse members. Where continuity of reinforcement is required, plain bars shall be lapped not less than 40 times their diameter; deformed bars not less than 30 times their diameter. Where bars are fabricated by positive welding at all joints, the lap may be of such length as will permit end cross bars to overlap each other by at least two in. The same requirements apply to end laps and side laps. All intersections of longitudinal and transverse bars shall be securely wired or clipped or welded together so as to resist displacement during the concreting operations.

(3) *Dimensions of Wire Fabric and Bar Mats.* The width and length of fabric sheets or bar mats shall be such that, when properly placed in the work, the extreme longitudinal and transverse members will be located approximately three inches from the edges of the slab, unless otherwise shown on the plans.

(4) *Bars, Chairs, and Stakes.* (*a*) *Bars.* Plain or deformed bars shall be made by either the Bessemer, the open-hearth, or the electric furnace process. Rolled rail steel will be accepted, except for tie bars that are to be bent and restraightened during construction. Their properties, chemical, tensile, bend-test, etc., shall conform to the requirements of the current Standard Specifications for Billet-Steel

Bars for Concrete Reinforcement, of the A. S. T. M. (Designation: A15) or for Rail-Steel Bars for Concrete Reinforcement, of the A. S. T. M. (Designation: A16). Bars depending upon bond for their effectiveness shall be free from excess rust, scale, or other substance which prevents the bonding of the concrete to the reinforcement.

(*b*) *Chairs.* The chairs for holding and maintaining tie rods in correct position while the concrete is being placed shall be of the following design, or equal:

The chairs shall be of metal of at least 14 gauge thickness, 12 in. long, slightly rounded, 1¾ in. wide measured along the metal, and tapered at one end.

(*c*) *Stakes.* Stakes used to support and maintain expansion joint fillers shall be of the U-shape, at least ¾ in. wide, ⅜ in. deep and of material not less than 16 gauge in thickness, or equal. The stakes shall be at least 17 in. long and of sufficient additional length to insure proper bearing support.

(*d*) *Dowel and Tie Bars.* Deformed steel bars meeting the specifications for bar reinforcement shall be used for the tie bars joining slabs across longitudinal hinged joints or joining slabs to curb or to gutters. Dowels across expansion joints, when specified, shall be smooth, round bars. Dowel and tie bars shall be of the design and placed as shown on the plans.

MISCELLANEOUS MATERIALS

(1) *Subgrade Paper.* Subgrade paper shall conform to the requirements of the current Standard Specification for Subgrade Paper, M74, of the A. A. S. H. O.

(2) *Expansion Joint Filler.* Expansion joint filler shall conform to the requirements of the current Specifications for Preformed Expansion Joint Fillers for Concrete, of the A. S. T. M. (Designation: D544).

(3) *Metal Contraction Joint Plate.* Plates for tongue and groove joints when used in longitudinal joints or transverse contraction joints shall consist of a sheet of steel or iron not thinner than 16 gauge. (U. S. Standard Gauge, 1893.) Each section shall be a continuous strip of metal not more than 15 ft. long and having a width ½ in. less than the depth of the pavement. It shall be of the cross-section shown on the plans and be provided with an end connection which will hold the ends of strips firmly together. It shall be punched for dowels, or tie bars, and pins, as shown on the plans.

Channel shaped pins pressed out of sheet steel of not less than 12 gauge (U. S. Standard Gauge, 1893) and not less than 15 in. long shall be used to hold the joint metal upright on the subgrade.

(4) *Dowel Sleeves.* When metal sleeves are used they shall cover the end of the dowel bar for a distance of not less than two (2), nor more than three (3) in. One end of the sleeve shall be closed so that concrete cannot enter. The sleeve shall have a suitable flange to hold the end of the sleeve at least one inch from the end of the bar during construction, to insure subsequent free movement of dowel in the sleeve. The sleeve shall be of such rigid design that the closed end will not collapse during construction.

(5) *Cover Materials for Curing.* (*a*) *Burlap.* Burlap shall be made from jute or hemp, and shall be in good condition, free from holes, dirt, clay, or any substance which would interfere with its absorptive qualities. It shall weigh not less than seven ounces per sq. yd. when clean and dry. Burlap may be made up into mats if proper care is used to handle the covering without marring the finished surface of the concrete. When burlap is dipped in water or is sprayed with water, it shall readily absorb the water and become thoroughly saturated.

(*b*) *Cotton or Jute Mats.* Mats for curing concrete shall be one-ply sheets of cotton or jute felt or other approved filler suitably covered on both sides with burlap, weighing not less than seven ounces per sq. yd. when dry. The filler shall weigh not less than eight ounces per sq. yd. when dry. Mats shall be satisfactorily stitched or tufted to prevent displacement of the filler.

2—Construction

GRADING

Subgrade Preparation and Setting Forms

General. The grading and preparation of earth subgrades for concrete pavements and pavement bases shall conform to the requirements of these specifications.

Grading Stakes. The extent of the necessary grading is shown approximately on the plans which form a part of the contract. After stakes have been set by the Engineer, it shall be the duty of the contractor and his employees to protect them. Stakes which become disturbed will be reset by the Engineer at the contractor's expense.

Disposal of Excavated Material. Material removed in excavation shall be used in fills on the job, if, in the opinion of the Engineer, it is suitable for this work. If excavation is in excess of fill, the excess material shall be disposed of by the contractor at his expense.

Excavation Below the Subgrade. Where the work is in cut, the contractor, in general, will be expected to excavate only to the true sur-

face of the subgrade. If the contractor excavates below the true surface of the subgrade as shown on the plans, he shall refill with approved material. The refilling material shall be compacted to the true surface in a manner satisfactory to the Engineer.

Borrowed Material. If filling is in excess of excavation, the contractor shall secure the necessary material from such borrow pits as may be indicated on the plans accompanying this contract, or if no such location is indicated or if the amount in the indicated location is insufficient he shall furnish material satisfactory to the Engineer. Payment shall be made for such material at a contract price per cubic yard.

Removal of Sod. Any sod within the limits of the fill shall be completely removed to a depth of at least six inches by means satisfactory to the Engineer.

Filling in Nine Inch Layers. All fills shall be made in layers of not more than nine inches thick and shall be thoroughly compacted by means of a self-propelled roller weighing not less than six tons. Inaccessible places and those places where rolling is not practical on account of possible damage to sub-surface structures, shall be thoroughly compacted with mechanical tampers capable of exerting a force in one blow equal to 250 lb. per sq. ft. The dead weight of any mechanical tamper shall be in excess of 40 lb. per sq. ft. of bearing surface. Payment for compacting of layers in fills by rolling or by tamping will be included in the price bid per cu. yd. for "Grading."

Removal of Unsuitable Material. Fill material shall not contain trash, brick, broken concrete, tree roots, sod or cinders, and all unsuitable material must be removed to a depth of at least 18 in. below the finished surface of the subgrade. When the contractor is directed to remove unsuitable material from the subgrade, he will be paid for removing it at the price bid for "Grading" and shall bring the surface to the correct elevation using approved material, placed and compacted as specified in these specifications. He will be paid for this work at the prices bid for grading, and materials used.

Removing and Grubbing for Stumps. Where fills are to be made to a depth greater than 10 ft., the ground shall be cleared of all underbrush and all trees felled and removed. Where fills are less than 10 ft. thick, all underbrush, trees, tree stumps and saplings shall be grubbed and entirely removed.

Solid Rock. Should solid rock (boulders of one cubic yard or more in content, or ledges in their original bed) be encountered, it shall be removed to grade and shall be estimated and paid for at a contract price per cubic yard.

Sidewalk Grading. Sidewalk grading included in the contract shall be done at the same time the roadway is rough graded.

Payment for Grading. The prices bid shall include the whole cost of excavating or filling as above specified, of grubbing of stumps necessary in carrying out the work, except as otherwise specified herein, of excavating and removing all materials of every description that may be encountered, unless otherwise specifically provided for herein, of shaping the subgrade to exact lines and grades given, of sloping cuts, intersections, and approaches, and of rolling or tamping below the subgrade in the case of fills; and in connection with street construction, the sloping and filling between the curb and sidewalk pavement, and the grading and shaping of sidewalk spaces where shown on the plans. The work will be estimated to the grades and slopes shown on the plans.

Subgrade Preparation

Rolling the Subgrade. Before the subgrade will be considered ready for foundation, it must have been compacted, with a self-propelled roller weighing not less than six tons. On inaccessible places where rolling cannot be carried out, the subgrade shall be thoroughly compacted with mechanical tampers capable of exerting a force in one blow equal to 250 lb. per sq. ft. The dead weight of any mechanical tamper shall be in excess of 40 lb. per sq. ft. of bearing surface. If after the subgrade has been rolled to the satisfaction of the Engineer and has been accepted by him, it is disturbed by hauling over it or in some other manner, it shall be restored to its condition at the time of acceptance by reshaping and rolling, without any additional compensation to the contractor.

Payment for Rolling. Payment for compacting the surface of the subgrade by rolling, or tamping in inaccessible places, as directed by the Engineer, shall be included in the price bid for grading, or material used.

Testing and Correcting Subgrade. The surface of the subgrade shall conform to the cross-section shown on the plans. Immediately prior to placing concrete the subgrade shall be tested for elevation by means of an approved template riding on the side forms. If necessary, material shall be removed or added, as required, to bring all portions of the subgrade to the correct elevation. It shall then be thoroughly compacted and again tested with the template. Concrete shall not be placed on any portion of the subgrade which has not been tested for correct elevation.

Wetting of Subgrade. Concrete shall be placed only on a moist subgrade except where subgrade paper is used in accordance with these specifications.

If concrete is placed directly on the subgrade, the subgrade shall be thoroughly wetted, as directed by the Engineer, a sufficient time in advance of the placing of the concrete to insure that there will be no puddles or pockets of mud when the concrete is placed. The amount of saturation shall depend upon the character of materials in the subgrade. The subgrade shall not be allowed to dry out before the concrete is placed.

Placing Subgrade Paper. After the subgrade has been shaped and compacted and before any concrete is placed, when specified, the subgrade shall be entirely covered with a layer of bituminous saturated paper conforming to the requirements of the current Standard Specification for Subgrade Paper, M74, of the A. A. S. H. O. Adjacent strips of paper shall lap 4 in. and the ends shall lap 12 in. After being placed on the subgrade, the paper shall be kept reasonably intact and shall not be hauled over nor have unnecessary holes punched through it.

Setting Forms

Forms. Side forms shall be made of metal of cross-section approved by the Engineer, and shall have a depth equal to the specified edge thickness of the concrete. Building up of forms will not be permitted. Flexible or curved forms of proper radius shall be used for curves of 100 ft. radius or less. Forms shall be of ample strength and shall be provided with adequate devices for secure setting so that when in place they will withstand, without visible springing or settlement, the impact and vibration of the finishing machine. In no case shall the base width be less than eight inches for forms eight inches or more in height. Flange braces must extend outward on the base not less than two-thirds the height of the form. The forms shall be free from warp, bends, or kinks. Any variation of the top of a form from its true plane shall not be greater than $\frac{1}{8}$ in. in 10 ft.

Grade and Alignment. Conformity of the alignment and grade elevation shown on the plans shall be checked, and necessary corrections made by the contractor immediately prior to placing the concrete. Where any form has been disturbed or any subgrade thereunder becomes unstable, the form shall be reset and rechecked.

Base Support. The subgrade under the forms shall be thoroughly compacted and cut to the true grade so that the form when set upon it will be uniformly supported for its entire length and at the speci-

fied grade. Any subgrade which at the form line is found below established grade shall be filled to grade in lifts of ½ in. or less for a distance of 18 in. on each side of the base of the form and thoroughly re-rolled or tamped. Imperfections and variations above grade shall be corrected by tamping or by cutting as necessary. In exceptional cases the Engineer may require suitable stakes driven to the grade of the bottom of the forms to afford additional firmness.

Staking Forms. Forms shall be staked by using not less than three pins for each 10 ft. section. A pin shall be placed at each side of every joint. Form sections shall be tightly joined by a locked joint free from play or movement in any direction.

Advanced Setting. Where practicable forms shall be set for at least 300 ft. in advance of the point where concrete is being placed. Forms shall remain in place at least 12 hours after the concrete has been placed against them and shall be cleaned and oiled each time they are used.

PLACING CONCRETE

General Requirements

The concrete shall be distributed to such depth, above grade, that when consolidated and finished, the specified slab thickness will be obtained at all points and the surface will not, at any point, be below the grade specified for the finished surface.

The concrete shall be deposited on the subgrade in such manner as to require as little rehandling as possible. It shall be thoroughly consolidated against and along the faces of the forms. Necessary hand spreading shall be done with shovels, not with rakes. Workmen shall not be allowed to walk in the concrete with boots or shoes covered with earth or other foreign substances.

Placing shall be continuous between transverse joints without the use of intermediate bulkheads.

Concrete shall be placed only on subgrade which has been prepared as hereinbefore prescribed and approved. At all times during operations, at least 300 ft. of subgrade shall have been prepared ahead of the mixer. Concrete shall not be placed on a frozen subgrade. No concrete shall be placed around manholes or other structures until they have been brought to the required grade and alignment.

Retempering concrete by adding water or by other means will not be permitted.

The pavement may be constructed either to its full width in a single construction operation or in single lanes, unless one or the

other method is expressly stipulated on the plans, with the exception that no pavement more than 30 ft. wide may be constructed in a single operation. When pavement is constructed in separate lanes, the junction line shall not deviate from the true line shown on the plans by more than ½ in. at any point and shall be tooled to the radius shown on the plans.

Placing and Finishing Concrete at Joints

The concrete shall be deposited on the subgrade at a location as near to the expansion and contraction joints as possible without touching them. It shall then be shoveled against sides of the joint simultaneously, maintaining equal pressure on both sides. It shall be deposited to a height of approximately two inches more than the depth of the joint, care being taken that it is worked under the load transfer devices. The concrete shall not be dumped from the discharge bucket of the mixer directly upon or against the joints, nor shall it be shoveled or dropped directly on top of the load transfer devices. In placing the concrete, against expansion and contraction joints and in operating the vibrator adjacent to them, employees shall avoid stepping upon or disturbing in any way the joints or load transfer devices, either before or after they are covered with concrete.

The concrete adjacent to the joint shall be compacted with a vibrator inserted in the concrete and worked along the entire length of the joint on both sides. The vibrator shall not come in contact with the joint, the load transfer devices, or the subgrade. The concrete shall be worked against the body of the joint at all points. If any of the dowel bars are displaced, they shall be realigned before the finishing machine passes over them.

After the concrete has been vibrated, the finishing machine shall be brought forward until the front screed is approximately eight inches from the joint. Segregated coarse aggregate shall be removed from in front of and off of the joint. The screed shall be lifted and brought directly above the joint, set upon it, and the forward motion of the finishing machine resumed. When the second screed is close enough to permit the excess mortar in front of it to flow over the joint, it shall be lifted and carried over the joint. Thereafter, the finishing machine may be run over the joint without lifting the screeds, provided there is no segregated coarse aggregate immediately between the joint and the screed or on top of the joint.

After the concrete has been placed on both sides of the joint and struck off, the installing bar shall be slowly and carefully withdrawn leaving the premoulded filler in place. After the installing bar or

the channel cap is completely withdrawn the concrete shall be carefully spaded, additional freshly mixed concrete worked into any depressions left by the removal of the installing bar. The installing bar shall be cleaned and re-oiled prior to each installation of a joint.

Immediately after all finishing operations, including brooming, have been completed and before the concrete has taken its initial set, it shall be edged adjacent to all expansion and contraction joints. The edging tool shall be manipulated so that a well defined, continuous radius is produced and a smooth, dense mortar finish obtained. The edging tool shall not be tilted while being manipulated.

After the removal of the side forms, the ends of premolded transverse joints at the edges of the pavement shall be carefully opened for the entire depth of the slab, and in the case of air cushion joints any concrete that has been deposited over the end closures shall be removed, care being taken not to injure the ends of the joint. After the curing period and before the pavement is opened to traffic, premolded joints in which the filler (or cap) does not come flush with the surface shall be sealed or topped out, leaving a neat uniform strip of an approved filler material.

Cold Weather and Night Concreting

Except by specific written authorization, concreting operations shall not be continued when a descending air temperature in the shade and away from artificial heat falls below 40° F. nor resumed until an ascending air temperature in the shade and away from artificial heat reaches 35° F.

When concreting is permitted during cold weather the temperature of the mixed concrete shall not be less than 60° nor more than 100° F. at the time of placing it in the forms. The aggregates may be heated either by steam or dry heat prior to being placed in the mixer. The apparatus used shall heat the mass uniformly and shall be so arranged as to preclude the possible occurrence of overheated areas which might injure the materials.

In no case shall the aggregates be heated to more than 150° F. Material containing frost or lumps of hardened material shall not be used.

When directed by the Engineer, concreting operations shall be discontinued due to insufficient natural light, unless an adequate and approved artificial lighting system is provided and operated.

When concrete is being placed during cold weather and the air temperature may be expected to drop below 35° F., a sufficient supply of straw, hay, grass or other suitable blanketing material shall be

provided along the line of the work and any time when the air temperature may be expected to reach the freezing point during the day or night the material so provided shall be spread over the pavement to a sufficient depth to prevent freezing of the concrete before it has thoroughly hardened. The period over which such protection shall be maintained shall be not less than five days. If required by the Engineer, concrete laid less than 24 hours shall also be covered by approved canvas or similar enclosures and devices capable of maintaining the temperature within the concrete at not less than 50° F. The contractor shall be responsible for the quality and strength of the concrete laid during cold weather and any concrete injured by frost action shall be removed and replaced at his expense.

Tolerance in Pavement Thickness

It is the intent of these specifications that the pavement shall be constructed strictly in accordance with the thickness shown on the plans. Where any pavement is found not so constructed, the following rules relative to replacement of the faulty pavement and adjustment of pavement shall govern:

For pavement slab, the average thickness of which, determined as hereinafter provided, is equal within one-eighth inch to the thickness required by the typical cross-section shown on the plans, the contract unit price bid shall be used in payment.

For pavement slab, the average thickness of which, determined as hereinafter provided, is less than the thickness shown on the plans by more than one-eighth inch, but by less than one-half inch, an adjusted unit price shall be used in payment, which price shall bear the same ratio to the contract unit price as the square of the average thickness of the slab bears to the square of the thickness specified on the plans.

No additional payment over the unit contract price bid will be made for any slab, should the average thickness of the pavement, determined as hereinafter provided, exceed the thickness shown on the plans.

The thickness of the slab will be determined by average caliper measurement of the thickness of cores taken from it. At such points as the Engineer may select, in each 1000 lin. ft. of pavement, two or more cores will be taken and measured. The average thickness of each full mile of slab, or any fraction of a mile, if the contract cannot be divided into an integral number of miles, will be determined from these measurements. An adjusted unit price shall be calculated for each mile or fraction and shall be used as the basis of pay-

ment for accepted yardage therein. In calculating the average thickness of the slab, measurements which are in excess of the thickness specified on the plans by more than one-eighth inch shall be considered as the specified thickness plus one-eighth inch, and measurements which are less than the specified thickness by one-half (½) inch or more shall not be included in the average.

Payment will not be made for slab which is found deficient in thickness one-half inch or more. Such pavement shall be removed and replaced by the contractor with pavement of the specified thickness at his entire expense for slab removal and replacement. When the measurement of any core indicates that the slab is deficient in thickness by one-half inch or more, determination shall be made of the actual thickness of transverse sections of the slab at 25-ft. intervals set off along the center line of the road in each direction from the affected location until, in each direction, a transverse section of the slab is found which is not deficient in thickness by as much as one-half inch. The area of slab for which no payment will be made shall be the product of the full width of the slab or strip of pavement multiplied by the sum of the distances in each direction from the affected location along the centerline of the road to the transverse sections found not deficient in thickness by as much as one-half (½) inch. If the contractor believes that the cores and measurements taken are insufficient to indicate fairly the actual thickness of pavement, he may request additional cores and measurements. Such measurements shall be made at intervals not less than two hundred (200) feet. The cost of additional cores and measuremeants shall be deducted from any sums due the contractor unless such measurements indicate that the slab within the area in question is of specified thickness.

INSTALLATION OF JOINTS AND REINFORCEMENT

General Provisions

Longitudinal joints and transverse joints shall be constructed in both concrete pavements and bases where and as indicated on the plans or in the special provisions.

Transverse joints shall be expansion joints, contraction joints, or construction joints. Unless other locations are shown on the plans, all longitudinal joints shall be along or parallel to the center line of the pavement. All transverse joints shall be at right angles to the center line and shall extend the full width of the pavement. All joints shall be perpendicular to the finished grade of the pavement and, when tested with a straight-edge, the adjacent slabs separated

by the joint shall not vary from a 10-ft. metal straight-edge by more than one-eighth inch. All joints shall be finished as shown on the plans or as indicated in the special provisions.

Especial care shall be taken to finish the subgrade accurately to the required cross-section, at the locations where expansion, contraction, or construction joints are to be installed. If, for any reason, the subgrade is trimmed too low, or if there are any open spaces beneath the joint, the joint shall be removed, the subgrade shall be backfilled and firmly tamped, and the joint reset.

The contractor will be required to exercise such care in placing, compacting, and finishing the concrete at and about all joints as to avoid the forming of honeycomb and voids. A suitable internal vibrator shall be used for compacting concrete along all forms and joints. Care shall be taken that the vibrating apparatus does not penetrate the subgrade.

The edges of the pavement at all joints shall be rounded to the radius indicated on the plans for the specific type of joint. All joints shall be continuous from form to form, from joint to joint, or from form to edge of slab, as the case may require. The final horizontal position of transverse joints must be at right angles to the centerline of the pavement on tangents and radially on curves. The final vertical position must be at right angles to the finished grade of the pavement. Longitudinal joints and reinforcement, except dowel bars, shall not extend through any transverse joints.

Longitudinal Joints

Longitudinal joints shall be installed as shown on the plans. Tie bars shall be placed across longitudinal joints as shown on the plans. Such tie bars shall be held in position, by chairs or other supports at both ends of the spacing shown on the plans or described in the special provisions. Tie bars shall be deformed steel bars of the diameter and length as shown on the plans or described in the special provisions.

Longitudinal "metal strip" joints shall be formed by first installing a metal parting strip of the required gage, shape and dimensions as shown on the plans, to be left permanently in place. The metal strip shall be held securely in place, true to line and grade, by steel pins, at intervals that average not greater than three feet, but in no case to exceed four feet. The minimum length of metal strips shall be 10 ft. and adjoining sections shall be securely fastened by lapping and pinning, by means of a slip joint, or other approved method. When so indicated on the plans, the metal strip shall be capped with

an approved installing shield to be removed after strikeoff and screeding. The contractor shall furnish an approved gage riding on the side forms for accurately checking the position of the parting strip before concrete is placed against it. In lieu of any proprietary deformed metal strip, a metal strip of other approved pattern may be substituted.

Longitudinal "dummy joints" shall be formed by depressing a groove or cleft in the slab of the dimensions shown on the plans. The groove, formed in the soft concrete by a suitable tooling device, shall extend vertically downward from the surface to the depth shown and shall be true to line. When the "joint" is of the poured type, the groove, as soon as possible after its completion, shall be cleaned and poured full of approved sealing material. When the "joint" is of the premolded type, strips of premolded filler shall be inserted in the groove immediately after it is formed. Longitudinal construction joints, that is, the joints between slabs placed separately, shall be formed as shown on the plans or as indicated in the special provisions. In forming these joints the contact edge (or edges) of the previously complete slab (or slabs) shall be painted with a heavy coat of bituminous material before the concrete of the adjacent slab is placed against it (or them), unless a premolded joint filler is required and used between the slabs. In such cases the joint shall be trimmed and sealed as required for transverse expansion joints.

Expansion Joints at Structures

Expansion joints shall be formed about all structures and features projecting through, into, or against the pavement. Unless otherwise indicated on the plans, such joints shall be not less than one-fourth inch thick and shall be of the premolded type.

Transverse Premolded Expansion Joints

Joints using transverse premolded expansion material shall be placed at such intervals as are shown on the plans. These joints shall be formed during the placing of the concrete and such methods of construction shall be employed that joints shall extend to the full depth and width of slab. The finished joint shall be at right angles to the subgrade. It shall be true to the transverse line prescribed within an allowable variation of one-quarter inch in the width of one traffic lane.

Transverse premolded expansion joints shall be formed by securely staking, perpendicular to the proposed surface of the pavement,

an approved "installing bar" or installing device, against which the premolded filler shall be fastened before placing the concrete. The installing bar shall be of substantial metal plate and shall have a length of one-half inch less than the required width of the slab and shall be cut to the required depth and crown of the slab and securely staked in position so that the top edge, unless otherwise provided for on the plans, shall be uniformly one-fourth inch below the proposed pavement surface. The lower edge shall be cut to conform to the prescribed cross-section of the subgrade. The installing bar shall be slotted from the bottom as necessary to permit the installation of the required dowels. Suitable means shall be provided on the bar for facilitating its removal. Header boards, sheet metal holders or other devices used in lieu of the installing bar must meet the approval of the Engineer. The joints shall be protected against damage from any cause whatsoever until such time as they are installed in the work. Any joints damaged during transportation, or by careless handling, or while in storage shall be replaced or repaired by the contractor. Repaired joints shall not be used until they have been approved by the Engineer.

The designated premolded joint filler shall be appropriately punched to the exact diameter and location of the dowels. It shall be furnished in lengths equal to one-half the designated width of the slab. Where more than one section is used in a joint, the sections shall be securely laced or clipped together. The premolded joint filler shall be placed on the side of the "installing bar" nearest the mixer. The bottom edge of the filler shall project to or slightly below the bottom of the slab, and the top edge, unless otherwise prescribed, shall be held about one-half inch below the proposed surface of the pavement in order to allow the finishing operations to be executed continuously. The top edge of the filler shall be protected while the concrete is being placed by a metal channel cap of at least ten gage material, having flanges not less than 1½ in. deep. The installing device may be designed with this cap self-contained.

Transverse Metal Strip Joints

Transverse "metal strip" contraction joints shall be formed during the placing of the concrete by installing a metal parting strip to be left in place. This strip shall conform to the requirements for type, gage, shape and dimensions shown on the plans or indicated in the special provisions, and shall be securely staked in place as shown on the plans. This strip shall be temporarily capped with a metal

channel cap. The dowel assembly and the method of placing it shall be as indicated for transverse expansion joints. Concrete shall be deposited as for transverse expansion joints. After the longitudinal floating has been completed but before the surface is broomed the metal cap shall be removed and the concrete edged, and the joint sealed as required for expansion joints.

Transverse Construction Joints

Unless other prescribed joints occur at the same points, transverse "construction" joints shall be made at the end of each day's run or where interruption of more than 30 minutes occurs in the concreting operations and where the length of pavement laid from the last joint is more than 10 ft. Sections less than 10 ft. long between joints will not be permitted. Transverse construction joints shall be laid in all particulars as prescribed for transverse metal strip "contraction joints."

Transverse Weakened Plane Contraction Joints in Pavement Base

Concrete bases shall be provided with expansion and contraction joints at spacings and of the type as shown on the plans. Unless otherwise shown on the plans contraction joints in bases may be made by forming a cleft or groove in the upper portion of the slab either by installation of subgrade paper, or other approved waterproof material, or by means of an installing bar or device used to form the cleft or groove. The installing bar shall be designed to remain straight and true during conditions of operation. The groove shall extend across the full width of the slab in a straight line and shall extend vertically downward from the surface for at least one-half the depth of the slab. The groove shall be formed before the concrete has taken an initial set and in such a manner that the finish of the concrete base shall not be unduly disturbed. Reinforcement shall not extend across weakened plane joints.

Note:—For bases other than those laid monolithic (such as brick vibrated into fresh concrete) with the surfacing material, it is especially important to reduce the interval between joints by means of closely spaced contraction joints so as to control the amount of joint opening (which may extend through the surface course) due to shrinkage and contraction. Experience indicates that the best results are obtained when the spacing of such joints does not exceed 20 ft. and preferably 15 ft.

Load Transfer Devices

Dowel bars or some other approved type of load transfer device shall be placed across all transverse joints unless otherwise indicated

on the plans. When dowel bars are used they shall be of the length, diameter, and spacing as shown on the plans. They shall be placed at the approximate center of the slab depth. Dowel bars shall be held in horizontal and vertical alignment by suitable multiple holders, or they shall be installed as preassembled units wherein the dowels are held in proper alignment by spacer bars welded or otherwise firmly secured to the fixed ends of the dowels.

The support stakes where used shall be located near the end of the dowel bars. They shall not be attached to the spacer bars, nor shall they be driven to support the dowel bars between the spacer bar and the expansion joints. They shall be left permanently in place.

The free end of each dowel bar shall be painted at the site of the work with one coat of red lead or basic sulphate blue lead paint. When the paint is dried and immediately before it is placed in position, the free end of each bar shall be thoroughly coated with heavy oil. The free end of each bar for expansion joints shall be provided with a close fitting dowel sleeve as hereinbefore specified.

When a load transfer device other than a dowel bar is used it shall be of a type, size, and spacing as shown on the plans, and shall be installed in a manner meeting the approval of the Engineer.

Installation of Joints

If the paving mixer is operated from the shoulder, the joints shall be set immediately after the final testing of the subgrade has been completed. If the paving mixer is operated from the subgrade, the joints shall be set immediately after it moves forward, so as to permit as much time as possible for proper installation.

After the subgrade has been prepared as provided hereinbefore, the assembled joint shall be lifted carefully into place on the subgrade. It shall be placed at right angles to the centerline of the pavement by means of the large triangle or other suitable methods. The top of the joint shall be set the proper distance below the pavement surface and the elevation checked by a properly designed templet. On widened curves, the longitudinal center joint shall be placed so that it will be, as nearly as possible, equidistant between adjacent wing anchor units when joints equipped with them are used. The joint shall be accurately set to the required line and grade and shall be securely held in the required position by stakes or an approved installing device or both throughout the operation of placing and finishing the concrete. Depending upon their design and upon the

direction of advance of concreting operations, the joint shall be installed so that the concrete pressure will not disturb it from proper alignment. No concrete shall be placed unless the methods and devices used by the contractor for installing, securing, and finishing the joints meet with the approval of the Engineer. The joints shall be vertical and no joint shall deviate more than one-fourth inch in horizontal alignment either way from a straight line. If joints are constructed in sections, there shall be no offsets between adjacent units. Dowel bars shall be checked for exact position and alignment as soon as the joint is staked in place on the subgrade and the joint shall be tested to determine whether it is firmly supported. Any joint not firmly supported shall be reset.

Placing Reinforcement

The concrete shall be reinforced if and as designated on the plans. All reinforcing steel, when received on the job, shall be held in approved storage and then shall be distributed along the work as approved by the Engineer. When placed in the work, it shall be free from dirt, dust, scale, or other foreign matter, and rust of such degree of development as will impair bond of the steel with the concrete.

When bar assemblies are shown on the plans, the reinforcement shall be assembled accordingly, and firmly fastened together at all intersections. All adjacent ends shall lap not less than forty diameters. Bar assemblies shall be placed and secured as shown on the plans. No displacement during concreting operations will be permitted.

When steel fabric is required it shall be placed at the depth shown on the plans. Sheets shall be lapped and shall clear all edges of the slab as shown on the plans. Where steel fabric is used, concrete shall be struck off by means of a templet at the indicated depth of the reinforcing below the finished surface of the slab. The fabric reinforcement shall then be placed directly upon the concrete so that it will remain in place, after which the balance of the required concrete shall be placed above it. All sections of mesh reinforcement shall be tied at the laps so as to be held securely in place.

Marginal bars shall be placed if and as shown on the plans on chairs or supports, as detailed in the plans or described in the special provisions to insure that the bars will be in their specified position after concrete is placed. Marginal bars shall be given one coat of red lead or of basic sulphate blue lead paint. At the time the marginal bars are placed, the paint shall be dry and hard. After being placed in position, the marginal bars shall be coated with heavy oil.

FINISHING

Finishing Methods

(*1*) *Machine Finishing.* The concrete as soon as placed shall be struck off accurately and screeded, by an approved finishing machine, to the crown and cross-section shown on the plans and to such elevation slightly above grade that when properly consolidated and finished the surface of the pavement shall be at the exact grade elevation indicated by the plans and free from porous places.

The finishing machine shall be of the screeding and troweling type, designed and operated both to strike off and to consolidate. The machine shall go over each area of pavement as directed, as many times and at such intervals as necessary to give the proper compaction and to leave a surface of uniform texture, true to grade and contour. At least two trips will be required. However, if necessary to insure the required density and finish, additional screeding will be required. Prolonged operation over a given area shall be avoided. The last trip for a given area shall be a continuous run of at least forty (40) feet. The top of the forms shall be kept free from accumulations by an effective device attached to the machine and the travel of the machine on the forms shall be maintained true without lift, wobbling, or other variation tending to affect the precision of finish. The finishing machine shall be of ample strength to withstand severe use and shall be fully and accurately adjustable for loss of crown or other derangement due to wear.

During the first pass of the finishing machine there shall be maintained a uniform ridge of concrete at least three inches in depth ahead of the front screed for its entire length. Except when making a construction joint, the finishing machine shall at no time be operated beyond that point where the above described surplus can be maintained ahead of the front screed.

(*2*) *Hand Finishing.* Where hand finishing is permitted the concrete shall be struck off and consolidated by means of a metal shod screed to the crown and cross-section shown on the plans and to such elevation above grade that when consolidated and finished, the surface of the pavement shall be at the required grade elevation. The screed shall be moved forward with a combined longitudinal and transverse motion, moving always in the direction in which the work is progressing and so manipulated that neither end is raised from the side forms during the strike off process.

(*3*) *Vibrated Finishing.* When a vibrated finishing is required, the finishing machine shall be equipped with means for applying high

frequency vibration to the upper surface of the concrete in accordance with the requirements specified for either of the following types. The particular type selected by the contractor shall be approved by the Engineer. The vibratory units shall be synchronized and shall operate at a frequency of not less than 3,500 cycles per minute.

(*a*) *Vibrating Units Mounted Directly on the Screeds.* This type shall consist of two independently operated screeds. The front screed shall be equipped with not less than one vibrating unit for each 7½-ft. length or portion thereof of vibratory screed. The front screed shall be not less than 18 in. wide and shall be equipped with a bull-nose front edge having a radius of not less than two inches.

(*b*) *Vibrating Pan Mounted Independently of the Screeds.* This type shall consist of two independently operated screeds together with an independently operated vibratory pan (or pans). The pan shall be mounted in such a manner as not to come in contact with the forms. It shall rest directly on the concrete and shall be so adjusted as to produce uniform vibration over the entire width of pavement surface. The pan shall be equipped with not less than one unit for each six feet of length or portion thereof of vibrating pan. The screeds shall be capable of operating in a position that will strike off the concrete at a sufficient height above the top of the forms to allow for proper compaction with the vibratory pan.

Note—Internal vibrators are now being studied and may be approved for use with standard finishing machines.

(*4*) *Floating.* As soon as possible, after the concrete has been struck off and consolidated, it shall be further smoothed and consolidated by means of a longitudinal float of a suitable design as approved by the Engineer. In this operation the longitudinal float shall be worked with a sawing motion, while held in a floating position parallel to the road centerline and passing gradually from one side of the pavement to the other. Movements ahead along the centerline of the road shall be in successive advances of not more than one-half the length of the float.

(*5*) *Straightedging.* After the longitudinal floating has been completed and the excess water removed, but while the concrete is still plastic, the slab surface shall be tested for trueness with a straightedge. For this purpose the contractor shall furnish and use an accurate 10-ft. metal straightedge swung from handles three feet longer than one-half the width of the slab. The straightedge shall be held in successive positions parallel to the road centerline in contact with the surface and the whole area gone over from one side of the slab

to the other as necessary. Advance along the road shall be in successive stages of not more than one-half the length of the straightedge. Any depressions found shall be filled immediately with freshly mixed concrete, struck off, consolidated and refinished. High areas shall be cut down and refinished. The straightedge testing and refloating shall continue until the entire surface is found to be free from observable departures from the straightedge and the slab has the required grade and contour.

(*6*) *Belting.* After the straightedging and when most of the water sheen has disappeared and just before the concrete becomes nonplastic, the surface shall be belted with a two-ply canvas belt, having a width of not less than six inches and a length at least two feet greater than the width of the slab, or with an approved wooden belt. Hand belts shall have suitable handles to permit controlled, uniform manipulation. The belt shall be operated with short strokes transverse to the road centerline and with a rapid advance parallel to the road centerline.

(*7*) *Brooming.* As soon after the belting as the surplus water has risen to the surface, the pavement shall be given a broom finish with an approved steel broom, not less than 18 in. wide. The broom shall be pulled gently over the surface of the pavement from edge to edge. Adjacent strokes shall be slightly overlapped. Brooming shall be perpendicular to the center line of the pavement and so executed that the corrugations thus produced will be uniform in character and width, and not more than one-eighth inch deep. The broomed surface shall be free from porous spots, irregularities, depressions and small pockets or rough spots such as are, or may be caused by accidentally disturbing, during the final belting and brooming, of particles of coarse aggregate embedded near the surface.

(*8*) *Edging.* After belting and brooming have been completed, but before the concrete has taken its initial set, the edges of the slab shall be carefully finished with an edger of the radius required by the plans, and the pavement edge left smooth and true to line.

(*9*) *Final Surface Test.* The contractor shall be held responsible for the correct alignment, grade, and contour specified. Any high spots in excess of one-eighth inch and not more than one-quarter inch from the 10-foot straightedge shall be ground to the required cross-section by the contractor at his own expense. Where the deviation from the required cross-section exceeds one-quarter inch the pavement shall be removed and replaced by the contractor at his own expense.

CURING

First 24-Hour Period

The concrete shall be covered with two thicknesses of approved wet burlap weighing not less than seven ounces per square yard, cotton mats, or other approved material of highly absorptive quality, immediately after final finishing of the pavement surface for not less than 24 hours. The material shall be thoroughly wet when placed and shall be kept saturated by spraying until removed.

Final Curing

For completion of the curing the burlap or cotton mats may be left in place and kept saturated for 72 hours, or they may be removed at the end of 24 hours, and the concrete surface further protected in accordance with the current Standard Method of Curing Portland-Cement Concrete Slabs with Wet Coverings of the A. S. T. M., (Designation: C84), or for at least an additional 48 hours in accordance with any suitable method approved by the Engineer. Forms shall not be removed from freshly placed concrete until it has set for at least 12 hours.

APPENDIX

PROTECTION OF FINISHED PAVEMENT

Protection of Concrete

The contractor shall erect and maintain suitable barricades and, when so indicated in the special provisions, shall employ watchmen to exclude traffic from the newly constructed pavement for the period herein prescribed and these barriers shall be so arranged as not in any way to interfere with or impede public traffic on any lane intended to be kept open, and necessary signs and lights shall be maintained by the contractor clearly indicating any lanes open to the public. Where, as shown on the plans or indicated in the special provisions, it is necessary to provide for traffic across the pavement, the contractor shall at his expense construct suitable and substantial crossings to bridge over the concrete. Such crossings, as constructed, shall be adequate for the traffic and satisfactory to the Engineer.

Any part of the pavement damaged by traffic or other causes occurring prior to its final acceptance shall be repaired or replaced by and at the expense of the contractor in a manner satisfactory to the Engineer. The contractor shall protect the pavement against both public traffic and the traffic caused by his own employees and agents.

Handling and Facilitating Public Traffic

Normal, unimpeded use of the thoroughfare of which the proposed pavement is to be a unit, is of value to the public and it is, therefore, mutually understood, under the contract of which these specifications form a part, that for the special sections of the thoroughfare identified on the plans as requiring special traffic handling and

for the distances stated thereon, surfaced road lanes as indicated shall be made available by the contractor for unimpeded public traffic at all times, and maintained in proper condition throughout the construction period. These lanes shall be of the clear widths stated on the plans, and shall be kept entirely free from encroachment at the time by any equipment of the contractor or by any workmen or employees of the contractor or by storage or transportation of any materials intended for the work.

The scheme and sequence of construction of the several lanes, slabs and sections of pavement, including the sequence of the shifting of public lanes during progress of construction, shall be as given on the plans or as described in the special provisions. The scheme and sequence so prescribed shall be followed in all details by the contractor.

Where the edge of any stipulated public traffic lane is contiguous to an edge of the particular slab or lane being placed, the contractor shall provide, erect and subsequently remove, a substantial temporary guard fence, as shown on the plans along the prescribed dividing line and maintained there until the slab is cured and opened to traffic. The fence shall be such as will effectually prevent workmen from moving into or standing in the public traffic lane, or impeding traffic. Workmen and employees of the contractor shall not be allowed to be in the reserved public lanes or in any way to impede traffic at any time. The plan of operations and equipment of the contractor shall be such as to obviate any necessity for encroachment on the public traffic lane or lanes. Where so shown, special lanes for the contractor's trucks and similar vehicles shall be provided, separate from and not interfering with the prescribed traffic lanes. Where the clearance between public traffic lane and the contractor's operating equipment is restricted, special delivering equipment may be necessary, designed to deliver and depart within the width of the slab actually being placed without encroaching on any public lane. Such equipment shall be provided when and if required on the plans.

Except where a special bid price for "traffic handling" is required in the proposal and in the special provisions, all cost of handling and protecting traffic, of special equipment, of temporary road surfacing and its maintenance, of temporary guard fences and of other things to be provided or to be done under this paragraph, shall be at the expense of the contractor. Before making his bid the contractor shall expressly examine the site of these special traffic accommodations and give detailed consideration to the probable cost and extra expense to which he will be put, including the use of special equipment.

Opening to Traffic

Traffic will ordinarily be excluded from the newly constructed pavement for 14 days after the concrete is placed and may be excluded for a longer period if cross-bending tests indicate that it is advisable to extend this time. Cross-bending test specimens, prepared at regular intervals from the concrete as it comes from the mixer and cured under the same temperature, moisture and climatic conditions as the corresponding slabs of pavement, shall be employed as a means of fixing the time of opening the pavement to traffic. These beams will be tested by standard laboratory methods and when these specimens as tested indicate that the corresponding pavement has attained a modulus of rupture of not less than 450 p.s.i., the pavement shall be cleaned, the joints filled and trimmed and the pavement opened to traffic. If this strength is not attained in less than 14 days, the Engineer

may open the pavement to traffic at his discretion. If it is not attained within 14 days, the pavement shall be kept closed until this strength is attained. In all cases, before the pavement is opened to traffic, it shall be cleaned and the joints shall be filled and trimmed or topped out as herein required. The joint or line of separation between adjacent strips or slabs of concrete, when the pavement is constructed in strips or slabs shall be cleaned and filled with an approved sealing material.

Discussion of this report should reach the A. C. I. Secretary in triplicate by June 1, for publication in the* JOURNAL *Supplement concluding this Proceedings volume

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Design and Control of Concrete Paving Mixtures —Texas*

By E. B. Cape†

SYNOPSIS

This paper presents a method employed by the Texas Highway Department for concrete paving mix design and control based on absolute volume. The two types of specifications under which the Department operates are described, namely, the modified water-cement ratio specifications (arbitrary cement factor), and the strength or quality specification which places no limit on either the minimum cement factor or maximum water-cement ratio. The mix design and control employed with each of these specifications are explained and examples given.

Eight year records of the service behavior of numerous low cement factor pavements are presented as evidence that concrete pavement of proper quality is obtained when the concrete is designed by the absolute volume method under the strength and quality specification. This specification and design methods permit materials to be used with the greatest efficiency and therefore assure concrete of the desired quality at the lowest cost.

The Texas Highway Department has had two primary aims in the design and construction of concrete pavements. The first aim has been to secure concrete pavement of the proper quality for the required service and the second aim has been to secure maximum economy in obtaining pavement of the quality selected. Pavement of the proper quality must have strength sufficient to resist stresses due to loads, stresses due to restrained warping, etc., and durability sufficient to resist weathering, including freezing and thawing, etc. To secure maximum economy it is necessary that the specifications, the design and the control of paving mixtures be such that the contractor may select materials, equipment and methods which will result in the greatest possible saving.

*Received by the Institute Dec. 9, 1940.
†Materials and Tests Engineer, Texas Highway Department, Austin, Tex.

HISTORY

Prior to 1929—Prior to 1929 the Texas Highway Department constructed more than 1,400 miles of concrete pavement using a specification calling for a minimum of 6½ sacks of cement per cubic yard of concrete with arbitrary mix proportions of 1:2:3½. No limiting water-cement ratio was specified. Results were erratic. The strengths of 6-in. pavement cores varied from 3,000 to 7,000 p.s.i. Grading requirements for both fine and coarse aggregates were so narrow and arbitrary that the cost of materials was excessive.

1929 and early 1930—During 1929 and the early part of 1930, the Department introduced a modified water-cement ratio specification in which the cement factor was limited to a minimum of 5½ sacks per cu. yd. and the water-cement ratio to a maximum of 5 gal. per sack of cement. It was unnecessary to consider the specified strengths in the design since they were so much lower than would be obtained with the limiting water-cement ratio. Mix proportions were based on Abrams' fineness modulus water-cement ratio tables and curves. From a study of the daily paving reports, it was soon apparent that proper control of strength and workability was lacking and studies were begun to develop a more workable and accurate control. The Division of Management of the United States Bureau of Public Roads sent a group of engineers to Texas to study batch design and control. The assistance of these engineers was very valuable at the beginning of our study.

Latter part of 1930—Several methods of mix design were studied, among which was the Talbot-Richart mortar voids method. As the result of the studies, the Department adopted an absolute volume control employing the use of b/b_o as defined by Talbot and Richart. In the Texas design, however, the term b/b_o is referred to as the workability factor or coarse aggregate factor. The absolute volume design as employed by Texas is not controlled by mortar voids tests and its only connection with Talbot and Richart method is the use of b/b_o. The absolute volume design and control as adopted provided a simple and accurate control of the mix consistency, the yield, and the cement factor. This design method and adequate field laboratory equipment installed for each project, gave much more uniform strengths of both cylinders and beams.

1931 to date—The absolute volume design and control as developed and adopted in 1930 have proved sound and have given satisfactory results during the last ten years both on Federal and State projects. Since the strengths obtained under the modified water-cement ratio

specification in use on all Federal concrete paving projects were greatly in excess of those specified by the Public Roads Administration, the Texas Highway Department took the stand that the quality of the pavement was higher than necessary and decided on a number of experimental, low cement factor concrete paving projects to determine their service value. If they proved satisfactory, great savings would result from low cement factor projects begun in 1931. The Department now has complete records of service value over a long period. On projects as early as 1932 strength was specified without limiting either the minimum cement factor or maximum water-cement ratio. The Department also conducted a laboratory research on portland cement concrete from 1931 to 1939. Results furnished authoritative data on which the specifications were greatly improved for fine aggregate, coarse aggregate, and construction procedures. The work also developed the use of mineral filler in concrete for improving the workability and plasticity and to prevent bleeding.

SPECIFICATION—MODIFIED WATER-CEMENT RATIO

The Department operates under two general concrete paving specifications, the first of which is the modified water-cement ratio specification by which both the minimum cement factor and the maximum water-cement ratio are specified on all Federal projects, since the Public Roads Administration has questioned the advisability of a strength specification without limits on the cement factor and water-cement ratio. Strengths being obtained are always considerably in excess of the requirements and they vary greatly from job to job depending on the aggregates and the brand and type of cement used. This type of specification is uneconomical because it does not give the desired quality of pavement at the lowest cost.

SPECIFICATION—STRENGTH OR QUALITY

The other type of specification under which the Department operates is a strength or quality specification. It places no limit either on the cement factor or the water-cement ratio. The concrete is designed for the specified modulus of rupture of 6 x 6-in. beams at seven days. The cement factor and water-cement ratio are determined from pilot tests employing the cement and aggregates as proposed for use on the particular project. This assures pavement of the specified quality and permits maximum economy with the materials available. Durability is assured by the use of sound materials and as a result of dense concrete. Inasmuch as the Public Roads Administration will not participate in concrete constructed under

this type of specification, it can be used only on projects involving State funds.

ABSOLUTE VOLUME DESIGN

Before the absolute volume design is explained, it is essential that two terms peculiar to the Texas design be defined.

The first of these, the coarse aggregate factor, is the ratio of the volume of coarse aggregate measured in a saturated, surface-dry and loose condition to the absolute volume of the concrete in which the coarse aggregate is used. The coarse aggregate factor is, therefore, the bulk volume of coarse aggregate per unit volume of concrete. This term is practically identical with the Talbot-Richart term b/b_o.

The second term is the sand factor, defined as the ratio of the volume of fine aggregate measured in a saturated, surface-dry and loose condition to the absolute volume of mortar in which the fine aggregate is used. The sand factor is therefore the bulk volume of fine aggregate per unit volume of mortar.

In the absolute volume design one cubic foot of water is assumed to consist of $7\frac{1}{2}$ gal. weighing $62\frac{1}{2}$ lb. One gallon of water is assumed to weigh 8.33 lb.

With a modified water-cement ratio specification, a minimum cement factor and a maximum water-cement ratio are definitely specified. As the amount of mixing water allowed is sufficient to maintain proper workability of the mix, this is equivalent to specifying an arbitrary cement factor.

With a strength or quality specification, only the flexural strength of the concrete is specified. The water-cement ratio is determined to produce the specified strength and the cement factor is varied to maintain the proper workability.

The difference is readily seen. Usually a modified water-cement ratio specification or arbitrary cement factor, results in producing concrete which has greater strength than is necessary for its designed purpose and adds materially to the cost. The scientific, the practical, and the economical method produces concrete sufficiently strong for the work intended with a reasonable safety factor.

When concrete is designed under a modified water-cement ratio specification, the cement factor, the water-cement ratio, and the coarse aggregate factor must be known. The sand factor is unknown. The problem is to find the weights of the different ingredients required to produce a definite volume of concrete. When concrete is designed under the strength specification, the water-cement ratio, the coarse aggregate factor, and the sand factor must be known.

The cement factor is unknown. The problem is to find the weights of the different ingredients required to produce a definite volume of concrete. As both methods are based upon absolute volume, there is no fundamental difference in the two problems.

For convenience in the following explanation and discussion, the factors which enter mix design will be separated into two main groups. One group includes specific gravity, per cent voids, per cent solids, free moisture, and absorption. In a given stockpile the aggregate, whether coarse or fine, has a definite specific gravity, a definite voidage and at the time the mix is designed, a definite free moisture or absorption content. Thus in designing a mix these factors are represented by fixed values which, inherent to the aggregate, cannot be changed. These factors will be called "aggregate characteristics."

The other group includes these factors: cement, water-cement ratio, coarse aggregate, and sand. In mix design these may be varied for strength, workability, and economy. These factors will be called "design factors."

Regardless of whether the mix is designed for a modified water-cement ratio or a strength specification, three of the design factors are always known or must be determined. In a modified water-cement ratio specification, the cement factor and maximum water per sack of cement are specified and a coarse aggregate factor is selected which will give the consistency and workability desired in the mix. With the strength specification, the water-cement ratio is determined to give the quality concrete desired and the sand and coarse aggregate factors are determined to give the consistency and workability to produce the most economical mix.

Fig. 1 illustrates the following explanation, which is a synopsis of the one fundamental procedure of absolute volume design. The mix design involves only three major steps:

1. The design factors and the aggregate voids or solids are employed to proportion the ingredients by absolute volume for a one sack batch.

2. The aggregate characteristics other than voidage are employed to convert the various absolute volumes to the respective batch weights.

3. The batch weights for a one sack design are increased or decreased to produce the desired absolute volume of concrete.

MODIFIED WATER-CEMENT RATIO CONCRETE

The modified water-cement ratio specifications now used on Federal projects provide for a minimum cement factor of 5 sacks per cu. yd. and a maximum water-cement ratio of 6½ gal. per sack of cement.

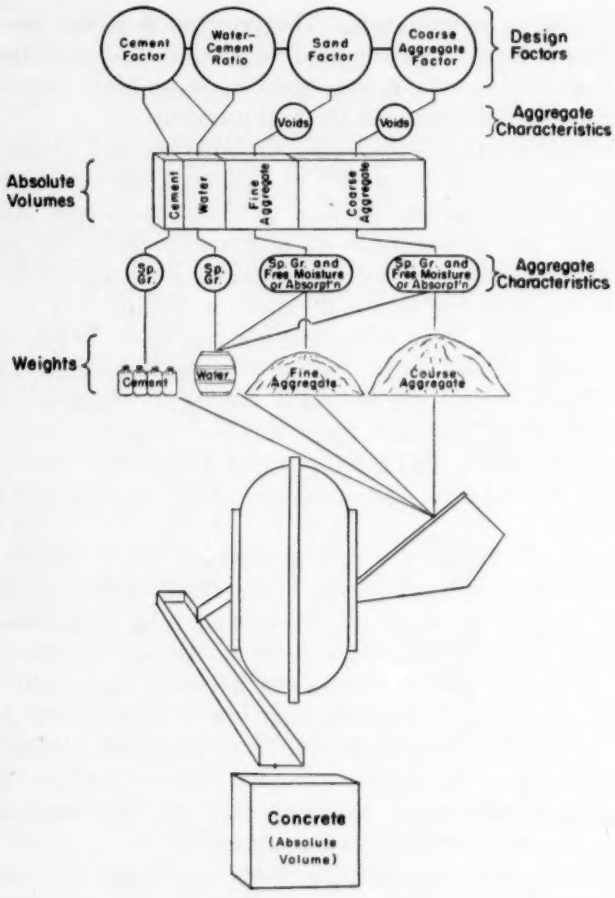

FIG. 1

Since strengths secured are adequate and need not be considered, the problem is to proportion the ingredients so that workable, cohesive and plastic concrete will be obtained having the specified cement factor and water-cement ratio. Before the mix design can be made, the apparent specific gravity, the unit weight, and the moisture content of each of the aggregates must be obtained.

Specifications limit the cement factor and water-cement ratio as follows:

Minimum cement factor = 5.00 sacks per cu. yd. of concrete
Maximum water-cement ratio = 6.5 gal. per sack of cement

Therefore, 6.5 x 5.0 = 32.5 gal. of mixing water per cu. yd. of concrete

The coarse aggregate factor is varied until the most workable and satisfactory mix is obtained, except that the specifications limit the factor to a maximum of 0.85. In this instance the mix will be designed initially with a coarse aggregate factor of 0.80.

The coarse aggregate has a specific gravity of 2.59, and solids of 62.2 per cent or voids of 37.8 per cent. The free moisture determination at time of use shows the coarse aggregate to contain 0.2 per cent free moisture.

The fine aggregate has a specific gravity of 2.62, and solids of 64.1 per cent or 35.9 per cent voids. The free moisture determination at time of use shows the fine aggregate to contain 2.1 per cent free moisture.

Summarizing, the design will be made for a one-sack batch of concrete with the following known design factors and aggregate characteristics:

Design factors:
- Cement factor = 5.00
- Water-cement ratio = 6.50
- Coarse aggregate factor = 0.80

	Aggregate Characteristics		Per Cent Free Moisture
	Per Cent Solids	Specific Gravity	
Cement	48.5	3.10	0.0
Coarse aggregate	62.2	2.59	0.2
Fine aggregate	64.1	2.62	2.1

A simple and logical procedure is as follows:

1. Determine yield or absolute volume of concrete produced by one sack of cement.
   One cubic yard of concrete must contain 5.00 sacks of cement
   One cubic yard contains 27 cu. ft.

   $$\text{Yield} = \frac{27}{5.00} = 5.400 \text{ cu. ft. per sack of cement}$$

2. Determine absolute volume of cement paste.
   One sack of cement weighs 94 lb.

   $$\text{Absolute volume of cement} = \frac{94}{62.5 \times 3.10} = 0.485 \text{ cu. ft. per sack}$$

   One cubic foot of water contains 7.5 gal.
   The water-cement ratio is 6.5 gal. per sack

   $$\text{Absolute volume of water} = \frac{6.5}{7.5} = 0.867 \text{ cu. ft. per sack}$$

   Absolute volume of cement paste = cement + water =
   0.485 + 0.867 = 1.352 cu. ft. per sack

3. Determine absolute volume of coarse aggregate.

   Two steps are necessary. First, the loose volume of coarse aggregate is determined. Since the coarse aggregate factor is 0.80, 0.80 cu. ft. of loose coarse aggregate will be used for each cubic foot of concrete. Since the yield is 5.400 cu. ft., loose volume of coarse aggregate is $5.400 \times 0.80 = 4.320$ cu. ft.

   Next, the absolute volume of coarse aggregate is determined. Since the solid content is 62.2 per cent, absolute volume of coarse aggregate is $4.320 \times 0.622 = 2.687$ cu. ft. per sack (absolute volume).

4. Determine absolute volume of fine aggregate.

   Absolute volume of fine aggregate equals absolute volume of concrete minus absolute volume of paste minus absolute volume of coarse aggregate $= 5.400 - 1.352 - 2.687 = 1.361$ cu. ft. per sack.

5. Check absolute volumes.

   Absolute volume of concrete obtained = absolute volumes of paste + fine aggregate + coarse aggregate $= 1.352 + 2.687 + 1.361 = 5.400$ cu. ft. per sack = yield.

Summarizing the calculations:

		Cu. Ft. Per Sack (Absolute Volume)
$\text{Yield} = \frac{27}{5.0}$	=	5.400
$\text{Cement} = \frac{94}{62.5 \times 3.10}$	=	0.485
$\text{Water} = \frac{6.5}{7.5}$	=	0.867
Paste $= 0.485 + 0.867$	=	1.352
Coarse Aggregate $= 5.400 \times 0.80 \times 0.622$	=	2.687
Fine Aggregate $= 5.400 - 1.352 - 2.687$	=	1.361
Concrete obtained $= 1.352 + 2.687 + 1.361$	=	5.400

The absolute volumes are converted to saturated, surface-dry weights by means of the respective specific gravities:

	Cu. Ft. (Abs. Vol.)	Lb. of Water Per Cu. Ft.	Specific Gravity	Lb.
Cement =	0.485	x 62.5	x 3.10	= 94.0
Water =	0.867	x 62.5	x 1.00	= 54.2
Fine Aggregate =	1.361	x 62.5	x 2.62	= 222.9
Coarse Aggregate =	2.687	x 62.5	x 2.59	= 435.0

The saturated, surface-dry weights of the fine aggregate and coarse aggregate are now corrected for the respective free moisture contents, and the designed amount of mixing water is compensated accordingly.

The fine aggregate contains 2.1 per cent and the coarse aggregate contains 0.2 per cent free moisture.

	Sat. Sur. Dry Wt. (lb.)		Correction $\left(1.0 - \frac{\% \text{ F.M.}}{100}\right)$		Batch Weight (lb.)
Fine Aggregate	222.9	÷	0.979	=	227.7
Coarse Aggregate	435	÷	0.998	=	435.9

Water in excess of the designed amount is being introduced into the mix with the aggregates in the form of free moisture:

Free water in fine aggregate	$= 227.7 \times 0.021$	= 4.8 lb.
(Check)	$227.7 - 222.9$	= 4.8 lb.
Free water in coarse aggregate	$= 435.9 \times 0.002$	= 0.9 lb.
(Check)	$435.9 - 435.0$	= 0.9 lb.
Total free water in aggregates	$= 4.8 + 0.9$	= 5.7 lb.
Weight of water to be added at mixer	$= 54.2 - 5.7$	= 48.5 lb.
Gallons of water to be added at mixer	$= \frac{48.5}{8.33}$	= 5.8 gal.

The sand factor for this mix may be calculated as follows:

$$\text{Sand factor} = \frac{\text{Volume of loose fine aggregate}}{\text{Absolute Volume of mortar}} = \frac{\frac{(1.361)}{(0.641)}}{0.485 + 0.867 + 1.361}$$

$$= \frac{2.123}{2.713} = 0.783$$

The final batch is therefore

Cement (lb.)	94
Fine Aggregate (lb.)	227.7
Coarse Aggregate (lb.)	435.9
Water (gal.)	5.8

These proportions are for a one-sack batch and will yield 5.400 cu. ft. of concrete. If a larger or smaller absolute volume of concrete is desired, multiply each of the above proportions by a factor determined by dividing the volume of concrete desired by the yield or, in this case by 5.400. Expressed algebraically,

$$\text{Factor} = \frac{\text{Yield desired}}{\text{Yield for one-sack batch}}$$

As stated previously, the coarse aggregate factor is varied until the most workable mix is secured. The foregoing design was based on the coarse aggregate factor of 0.80. It is apparent from the design that as the coarse aggregate factor is increased the absolute volume of coarse aggregate per cubic unit of concrete will be increased, and since the yield remains constant, the sand factor and absolute volume of fine aggregate will be decreased. By way of illustration, if the previous design for a one-sack batch had been based on a coarse aggregate factor of 0.82, the absolute volume of coarse aggregate in cubic feet would have been increased from 2.687 to 2.754 and the absolute volume of fine aggregate in cubic feet would have been decreased from 1.361 to 1.294. The saturated surface-dry weight of coarse aggregate would have been increased from 435.0 lb. to 445.8 lb., and the saturated surface-dry weight of fine aggregate would have been decreased from 222.9 lb. to 211.9 lb. The sand factor would have been decreased from 0.783 to 0.763.

STRENGTH OR QUALITY CONCRETE

Under a strength or quality specification, it is first necessary to determine the sand factor with the cement and aggregates proposed for use to give mixes of satisfactory workability for a wide range of water-cement ratios. The next problem is the determination of the water-cement ratio necessary with the proposed materials to give the specified strength. As with the modified water-cement ratio specification, it is necessary, before any mixes are made, to secure the apparent specific gravity, the unit weight, and the moisture content of each of the aggregates and to calculate the per cent of solids and the per cent of voids.

The information for proportioning mixes for the pilot strength tests is obtained by trial mixes, which may consist either of small concrete mixes or small mortar and concrete mixes. When mortar trial mixes are made only two water-cement ratios are selected, one well above and one well below the estimated water-cement ratio for the required strength. Small mixes are made by hand, varying the sand factor for each of the selected water-cement ratios until a mortar is obtained which will produce concrete of the consistency and workability desired. A sand factor curve is plotted from this information showing sand factor versus water-cement ratio. After a little practice it is not difficult to determine the best sand factor for use, that is, one which will produce a mortar of best workability and one which will not bleed. When concrete mixes are made they consist of small batches mixed by hand. Mixes are usually designed with a coarse aggregate factor (b/b_o) of from 0.80 to 0.82 using several water-cement ratios representing a wide range of values. Trial mixes are repeated on each water-cement ratio until a mix is secured having the specified slump and satisfactory workability and plasticity. A sand factor curve may also be plotted from the results of these mixes.

Using the proportions determined from the trial mixes, large mixes are made in a standard concrete mixer. The slump and workability of the concrete are checked in accordance with the specification requirements. At least 20 6x6x20-in. beams are made for each of the selected water-cement ratios. The beams are cured in water at the field laboratory and tested at the age of seven days. From the results of these tests a pilot strength curve is plotted showing modulus of rupture versus water-cement ratio in gallons per sack. From the pilot strength curve the water-cement ratio corresponding to the specified strength is selected.

Using the water-cement ratio selected from the pilot strength curve and the sand factor from the trial mix curve, a job mix is designed using a coarse aggregate factor (b/b_o) of 0.80 to 0.82 as follows: the coarse aggregate factor is varied after paving is started to give a mix of best workability, but the specifications limit this factor to a maximum of 0.85. Designs are usually started at intermediate values such as 0.80 or 0.82. Specifications limit nothing but the flexural strength of the concrete. Assume that the specifications for this particular job require a flexural strength of 500 p.s.i. at seven days and that the flexural strengths already obtained from the pilot strength curve show that a water-cement ratio of 9.7 gal. per sack will produce a concrete which will meet this specification.

Assume that the sand factor curve indicates a factor of 0.90 for a water-cement ratio of 9.7 gal. per sack of cement.

The coarse aggregate has a specific gravity of 2.66 and solids of 62.4 per cent or voids of 37.6 per cent. The absorption determination at the time of use shows the coarse aggregate to contain 0.4 per cent absorption.

The fine aggregate has a specific gravity of 2.64 and solids of 63.9 per cent or 36.1 per cent voids. The moisture determination at the time of use shows the fine aggregate to contain 3.5 per cent free moisture.

Summarizing, the design will be made for a one-sack batch of concrete with the following known design factors and aggregate characteristics:

Design factors:
- Water-cement ratio = 9.70
- Coarse aggregate factor = 0.81
- Sand factor = 0.90

	Aggregate Characteristics			Per Cent Abs.	Per Cent Free Moisture
	Per Cent Solids	Per Cent Voids	Specific Gravity		
Cement	48.5	51.5	3.10		
Coarse aggregate	62.4	37.6	2.66	0.4	
Fine aggregate	63.9	36.1	2.64		3.5

A simple and logical procedure is as follows:

1. Determine absolute volume of cement paste.

One sack of cement weighs 94 pounds

$$\text{Absolute volume of cement} = \frac{94}{62.5 \times 3.10} = 0.485 \text{ cu. ft. per sack}$$

One cu. ft. of water contains 7.5 gal.
Water-cement ratio is 9.7 gal. per sack.

$$\text{Absolute volume of water} = \frac{9.7}{7.5} = 1.293 \text{ cu. ft. per sack}$$

Absolute volume of cement paste = cement + water = 0.485 + 1.293 = 1.778 cu. ft. per sack

2. Determine absolute volume of mortar.

Two steps are necessary. First, the absolute volume of paste for one cubic foot of mortar is determined. Since the sand factor is 0.90, each cubic foot of mortar will contain 0.90 cu. ft. of fine aggregate, saturated, surface-dry and loose. Beginning with this 0.90 cu. ft. of fine aggregate and filling its voids with cement paste, the amount of paste will be $0.90 \times 0.361 = 0.3249$ cu. ft. (absolute volume). The remainder of the cubic foot of mortar is paste, or $1.00 - 0.90 = 0.10$ cu. ft. The total absolute volume of cement paste per cubic foot of mortar = 0.3249 + 0.10 = 0.4249 cu. ft., the remainder of the cubic foot of mortar being sand.

Next, the absolute volume of mortar which will be produced by one sack of cement is determined. Since one sack of cement will produce 1.778 cu. ft. of cement paste and since 0.4249 cu. ft. of paste will produce one cubic foot of mortar, 1.778 cu. ft. will produce $\frac{1.778}{0.4249} = 4.185$ cu. ft. of mortar (absolute volume).

3. Determine absolute volume of fine aggregate.

Fine aggregate = total mortar minus cement paste = 4.185 − 1.778 = 2.407 cu. ft. (absolute volume).

4. Determine absolute volume of concrete.

Two steps are necessary. First, the absolute volume of mortar per cubic foot of concrete is determined. Since the coarse aggregate factor is 0.81, each cubic foot of concrete will contain 0.81 cu. ft. of coarse aggregate, saturated surface-dry and loose. Beginning with this 0.81 cu. ft. of coarse aggregate and filling voids with mortar, the amount of mortar will be $0.81 \times 0.376 = 0.3046$ cu. ft. (absolute volume). The remainder of the cubic foot of concrete is mortar, or $1.00 - 0.81 = 0.19$ cu. ft. The total absolute volume of mortar per cubic foot of concrete = 0.3046 + 0.19 = 0.4946 cu. ft., the remainder of the cubic foot of concrete being coarse aggregate.

Next, the absolute volume of concrete which will be produced by one sack of cement is determined. Since one sack of cement will produce 4.185 cu. ft. of mortar and since 0.4946 cu. ft. of mortar will produce one cubic foot of concrete, 4.185 cu. ft. will produce $\frac{4.185}{0.4946} = 8.461$ cu. ft. of concrete per sack of cement (absolute volume).

5. Determine absolute volume of coarse aggregate.

The absolute volume of concrete minus absolute volume of mortar = absolute volume of coarse aggregate = 8.461 − 4.185 = 4.276 cu. ft. per sack of cement.

6. Check total concrete.

Total concrete = cement + water + fine aggregate + coarse aggregate = 0.485 + 1.293 + 2.407 + 4.276 = 8.461 cu. ft. per sack of cement (absolute volume) = yield.

7. Determine cement factor.

Since one cubic yard = 27 cu. ft., and since one sack of cement will produce 8.461 cu. ft. of concrete, cement factor = $\frac{27.0}{8.461}$ = 3.19 sacks of cement per cu. yd. of concrete.

Summarizing, absolute volumes in cubic feet for one-sack batch:

Cement	0.485
Fine aggregate	2.407
Coarse aggregate	4.276
Water	1.293

These absolute volumes are converted to weight in pounds, after which these weights are corrected for moisture content to give the bin weights for a one-sack batch. This procedure is identical with that given under "Modified Water-Cement Ratio Concrete" and will not be reported here.

Proportions by weight for one-sack batch (saturated surface-dry aggregates:)

Cement (lb.)	94.0
Fine aggregate (lb.)	397.2
Coarse aggregate (lb.)	710.9
Water (lb.)	80.8

Bin weights and water to be added at mixer for one-sack batch (corrections made for moisture):

Cement (lb.)	94.0
Fine aggregate (lb.)	411.6
Coarse aggregate (lb.)	708.1
Water (lb.)	69.2

The above weights for a one-sack batch will yield 8.461 cu. ft. of concrete. These proportions may be adjusted, as previously explained, to produce the absolute volume of concrete desired.

It is apparent from the foregoing design that the cement factor is a function of the voids in the coarse and in the fine aggregates, the cement factor increasing as the voids increase. It is also apparent that the cement-factor is a function of both the sand factor and the coarse aggregate factor, the cement factor decreasing as either or both of these factors increase. It is therefore imperative to keep both the sand factor and the coarse aggregate factor as high as possible, consistent with proper workability, to realize the greatest economy.

After the job has been started using the water-cement ratio selected from the pilot curve, no changes will be made in the water-cement ratio until the results from at least ten beam tests definitely show that the water-cement ratio employed is either too high or too low. No changes are made in the water-cement ratio so long as the average modulus of rupture does not vary from the specified modulus of rupture by more than 25 p.s.i.

CONTROL OF MIXTURES ON THE JOB

That the foregoing design and control be successful, it is imperative that it be directed by a trained personnel. The Texas Highway Department has conducted training schools for concrete inspectors to familiarize them with the design and control procedure. On each

concrete paving project the Department places a chief paving inspector, a plant inspector, a mixer inspector, and a surface inspector. The chief inspector correlates all of the design and control procedures under the direction of the Resident Engineer assigned to the project. The plant inspector designs all of the mixes and makes all of the necessary tests for control. He works in very close cooperation with the mixer inspector who passes on the workability and consistency of the paving mix at all times and is responsible for seeing that the mix meets specifications for workability. The surface inspector gauges the surface of the concrete and is under the direction of the mixer inspector.

The specifications of the Department require that an adequate field laboratory be constructed by the paving contractor. The laboratory is equipped by the Department with necessary half-gallon Mason fruit jars, a sand remixing bag, steel tapes, slump cones, one torsion balance, concrete aggregate scales, pycnometer tops, half cubic foot and one-tenth cubic foot measures, aggregate pans, depth test tools, one beam breaking machine, squares, small shovels, prescription bottles and a quantity of 3 per cent solution of Sodium Hydroxide, beam molds, trowels, tamping rods, cement sampling tubes, test weights for checking bins, coarse aggregate screens, sand sieves, armored cement thermometers, cylinder molds and base plates, and maximum-minimum thermometers. Additional equipment is furnished as needed.

The plant inspector makes moisture tests on all aggregates at least once each hour and corrects the batch weights accordingly. He makes unit weight tests at least twice each day, screen analyses on all aggregates, and apparent specific gravity tests at least once each week. The mixer inspector makes a slump test and two 6x6x20-inch beams for each 300 lin. ft. of pavement placed. He makes depth tests or soundings of the pavement for not more than 100 lin. ft. placed. Surface tests are made over the entire pavement surface by the surface inspector.

After beams are transferred to the field laboratory and are cured seven days, they are tested on the Highway Department beam machine equipped for center-point loading. The chief inspector and the plant inspector prepare paving reports giving complete job records of which Fig. 2 is a sample daily report.

ACCOMPLISHMENTS

Since 1930 the Texas Highway Department has constructed concrete pavement as follows:

Cement Factor *Sacks Per Cubic Yard*	*Barrels Per Cubic Yard*	*Length* (Miles)
Less than 3.50	Less than 0.875	56.920
3.50 to 3.99, incl.	0.875 to 0.998, incl.	86.589
4.00 to 4.49, incl.	1.000 to 1.123, incl.	554.482
4.50 to 4.99, incl.	1.125 to 1.248, incl.	33.712
5.00 to 5.49, incl.	1.250 to 1.372, incl.	2505.924
5.50 and greater	1.375 and greater	841.188

Form No. 312—Rev.

TEXAS STATE HIGHWAY DEPARTMENT
FIELD LABORATORY REPORT FOR CONCRETE PAVEMENT

County Monroe A. P. No. 25 Hwy. No. 432 Weather Clear & Windy Date 5/22/33

(SEE DESIGN)	DESIGN NO. 3				DESIGN NO. 4				DESIGN NO. 10				DESIGN NO. 11			
Materials	Sp. Gr.	D. L. Wt. Cu. Ft.	% Solids	A. V. Pas Sk. Cem.	Sp. Gr.	D. L. Wt. Cu. Ft.	% Solids	A. V. Pas Sk. Cem.	Sp. Gr.	D. L. Wt. Cu. Ft.	% Solids	A. V. Pas Sk. Cem.	Sp. Gr.	D. L. Wt. Cu. Ft.	% Solids	A. V. Pas Sk. Cem.
Water		1.0		.773				.773	Water 3.5 Sx			.467	1.0			.733
Cement		3.10	94.0	48.5	.485			.485	Cement 7 Sx			.485	3.10	94.0	48.5	.485
1 Fine Agg.		2.63	103.5	63.0	1.413			1.345				.859	2.63	103.0	62.7	1.107
2 " "													2.64	100.6	61.0	.368
C "		2.64	103.0	62.4	2.729			2.797				2.046	2.65	101.2	61.1	2.706
Totals			WF 81		5.400	Change WF to 83		5.400		WF 85		3.857				5.400
													Use 25% fine sand			

DESIGN FOR 6 & 8 4 BACK BATCH— 32.4 CU. FT.

SLAB REPORT Mixed sand unit wt 106.0

Time Batch	Des. No.	Cement Lbs.	FINE AGGREGATE Wt. D. L.	FINE AGGREGATE Moist. Lbs.	FINE AGGREGATE G/F	COARSE AGGREGATE Wt. D. L.	COARSE AGGREGATE Moist. Lbs.	COARSE AGGREGATE W/F
6:10	3	564	1394	60	84	2702	19	81
7:-	"	"	"	64	"	"	24	"
7:30	"	"	"	61	"	"	22	"
(Continuing on 7:00 test)								
8:15	"	"	"	64	"	"	24	"
9:-	4	"	1327	60	82	2769	28	83
10:-	"	"	"	62	"	"	"	"
11:05	10	790	1186	55	75	2836	91	85
12:-	4	564	1327	64	82	2769	28	83
1-	"	"	"	65	"	"	25	"
2:30	"	"	"	67	"	"	25	"
3:15	11	564	1032	54				
			365	16	85	2689	27	82
4:00		"	"	55/15	"	"	24	"
5-	3	564	1394	64	84	2702	27	81

Time Batch's	Sta. to Sta.	Slab No.	Length Ft.	Batches	Water Added Gals.	Total Mixed Gals.	Total Water Gals.	Cem. Fact.	Water Per Sack	Depth
										in ½ ths
6:25	132+12-132+41	112-B	29	9	25	9.5	34.5	5.0	5.8	
	-133+59	3	118	37	"	"	"	"	"	6 6 5T 6
7:25	-134+76	4	119	"	24	10.6	34.6	"	"	6 6 6 6
"	Last-did not reach mixer									
	-135+20	5	42	13	"	"	"	"	"	6' 6 6 6
8:35	-135+96	"	76	24	"	"	"	"	"	
	-136+28	6	32	10	"	"	"	"	"	6 5T 6 6
9:20	-137+15	"	87	27	"	"	"	"	"	
	-138+33	7	118	37	"	"	"	"	"	6 6 6 6
10:50*	-139+17	8	84	26	"	10.8	34.8	"	"	
11:25	-139+77	9	60	19	19	10.3	29.3	7.0	3.5	6 6 6 5T
	140+88-141+48	120	"	18	"	"	"	"	"	
12:20	-142+66	1	118	37	24	11.0	35.0	5.0	5.8	6 6 6 6
	-143+02	2	36	11	"	"	"	"	"	
1:25	-143+41	"	39	12	"	10.8	34.8	"	"	
Const. joint - Shut down to repair fin. machine 45 min.										
	-143+85	"	44	14	25	"	35.8	"	6.0*	6 6 6 6
2:45	-145+03	3	118	37	24	11.0	35.0	"	5.8	6' 6 6 6
3:30	-146+22	4	119	"	21.5	11.6	33.1	"	5.5	6 6 6 6
4:15	-147+40	5	118	"	"	11.3	32.8	"	"	6' 6 6 6'
	-147+73	6	33	10	"	"	"	"	"	
5:20	-148+59	"	86	26	24	10.9	34.9	"	5.8	6 5T 6 6
	-148+98	27A	39	12	"	"	"	"	"	

BEAMS AND CYLINDERS MADE				BEAMS BROKEN			
Sta. No.	Beam No.	Cyl. No.	Slump	No.	72 hour	No.	28 D'y
133	45	45	1½	38A	505	5	740
136	46	46	"	" B	480	6	765
139+30	47A	47	1	" C	490	7	750
" +60	" B		"		7 day		
141	" C		"	24	645		
" +30	" D		"	25	680		
143	48	48	1½	26	665		
146	49	49	"				

CEMENT Two Moon FROM Possumwalk

Time	°F. Chg.	°F. Air	Chg. Car No.	Beam No.	No. Bx.	Sta. to Sta.
6:	60	45	DT43542	15892-3	780	132+12-136+28
8:45	80	60	B076287	15894-5	1139	-143+02
12:30	85	65	Trucks	Sample 14	1110	-148+98
3:30	80	55				
5:30		50				

Pvt. 1575 L.F. 3150 SY. % in place 42

Bulk Cem. Used 3029 Bbls. 3023 Over Grades 6sx

Batch bar 8872 Bbl. mat. — Bbl.

Length haul 2.1 mi. No. Trucks 22-Single

Min Time 61 Sec. | Hrs. 11 Min. 0:45 last.

Time last Slab finished 6:45 - last batch 6:10

Remarks: * Delayed - truck had flat.

139+77-140+88 Br exception. 7sx in approach slabs for high early strength.

No widening. Subgrade excellent.

Extra water allowed for losses.

Loaded trucks waited at mixer one hour.

Source of Water and Aggregates:

Water - Blister Creek Sta. 140

1 Sand - Alto pit, Soprano, Tex.

2 " - Base " Barilone, "

Gravel - Alto " Soprano, "

Inspector Chico Hunbuson

B-160—1232—10m. NO. 75

FIG. 2—SAMPLE DAILY REPORT

TABLE 1—COMPRESSIVE STRENGTH OF 6-INCH CORES TAKEN FROM VARIOUS PAVEMENT SLABS OVER VARIOUS TIME INTERVALS

Project Number County	Cement Factor (Sks)	Original Cores			Additional Cores				Additional Cores				Additional Cores			
		No.	Av. Stress (psi)	Date Cored	No.	Av. Stress (psi)	Date Cored	Gain in Str. (%)	No.	Av. Stress (psi)	Date Cored	Gain in Str. (%)	No.	Av. Stress (psi)	Date Cored	Gain in Str. (%)
SP-767C Carson	4.00	45	4360	3/'32	45	4850	11/'34	11.2	17	5177	12/'36	18.7	9	5950	6/'40	36.5
SP-767D Potter	4.00	11	4715	7/'32	11	5077	11/'34	7.7	4	5580	12/'36	18.3	8	5710	6/'40	21.1
SP-237-1A Hill	2.00	3	1064	7/'34	3	1291	7/'35	22.2	3	1593	1/'37	49.7	3	1890	11/'38	77.6
SP-960A Grayson*	3.00	3	2535	1932	3	3297	1935	30.0	3	3213	1/'37	26.7	3	3170	1940	25.0
SP-960A Grayson*	3.50	3	3527	1932	3	5657	1935	60.3	3	4650	1/'37	44.7	3	5200	1940	47.4
FAP-633B Guadalupe	5.00	7	5185	8/'33	7	5730	2/'35	10.5	8	5327	8/'35	2.7	10	5825	8/'39	12.3
SP-974D Cameron	2.8 to 3.20	5	3342	8/'32	5	4888	10/'35	46.2	5	5092	11/'36	52.3	5	5240	5/'40	56.8
SP-956A Nueces	3.60	3	4307	9/'31	3	4810	8/'34	11.7	2	5225	11/'36	21.3	3	5265	7/'40	22.2
SP-956A Nueces	2.00	4	1780	9/'31	4	2480	8/'34	39.3	3	3163	11/'36	77.7	4	3550	7/'40	99.4
SP-556A Shelby	3.3 to 3.45	23	3293	7/'35	23	4292	12/'36	30.3	5	5054	11/'38	53.5	5	4982	4/'40	51.3
SP-556B Shelby	3.4 to 3.90	22	3971	7/'35	22	5209	12/'36	31.2	5	5556	11/'38	39.9	5	5306	4/'40	33.6

(*)-Rolled concrete—experimental.

Table 1 shows a record of eleven projects which have been cored from time to time showing conclusively that none of the concrete pavement has suffered loss in strength from natural weathering, including alternations of freezing and thawing, although the water-cement ratios of many of the projects are considerably higher than those ordinarily used in most other states.

TABLE 2—EXTENT AND NATURE OF SURFACE WEAR ON EIGHT SELECTED PROJECTS

Project Number	County	Year Built	Approx. Length Miles	Cement Factor Sacks Per Cu. Yd.	Percentage of the Locations Surveyed Which Were Classified as Indicated					
					A	A—	B	B—	C	D
SP-886A II	Live Oak	1932	14	3.5	10	62	26	1	1	0
SP-931ABC	Live Oak	1933	15[1]	3.1[2]	1	44	49	5	1	0
SP-840A	Refugio	1932	18[3]	3.7[4]	5	42	53	0	0	0
SP-840D II	Refugio	1933	18	4.0	19	59	21	1	0	0
FAP-543D II	Victoria	1930	15	5.0	58	38	4	0	0	0
SP-556AB	Shelby	1935	8	3.3[2]	27	52	21	0	0	0
SP-767D	Potter	1932	17	4.0	33	66	1	0	0	0
SP-767C	Carson	1932	17	4.0	34	58	8	0	0	0

[1]Includes portion of road within city limits of Three Rivers and George West not included in this classification.
[2]Average value. Design mix.
[3]Includes portion of road within city limits of Refugio not included in this classification.
[4]Average value. Varied from 3.6 to 3.8.

A description of the classifications used in grading the surface wear on concrete pavements is as follows:

Grade A represents pavement which has retained the herringbone finish and steel trowel and belt marks from the original finishing operations.

Grade A− represents pavement which has retained the original mortar finish, but where the herringbone texture has disappeared in occasional sections of the wheel lanes as a result of the abrasion from traffic.

Grade B represents pavement where the abrasion from traffic has worn away the light layer of mortar brought to the top in finishing operations sufficiently to expose the coarse aggregate in the concrete in the wheel lanes.

Grade B− represents pavement where the thin layer of surface mortar has disappeared over the major part of the slab surface sufficiently to expose the coarse aggregate.

Grade C represents pavement where the surface mortar has disappeared from practically all of the slab surface sufficiently to expose the coarse aggregate.

Grade D represents pavement which has worn sufficiently to permit the coarse aggregate to become loose and show signs of distress or disintegration.

Table 2 shows the extent and nature of surface wear on the eight selected projects as a summary prepared from notes taken during a joint survey made by the Public Roads Administration and the Texas Highway Department in April, 1939. The classification of each of these projects was also checked by representatives of the Public Roads Administration and the Texas Highway Department in Aug., 1940.

It is believed that this tabulation represents the average surface of the concrete pavements in the State of Texas There is no question that the surface wear on the two low cement factor base course projects in Live Oak County, listed as one and two in Table 2, is more severe than on any of the remaining projects in Texas. It should be stated, however, that the materials were produced largely from roadside pits and, since projects of this nature were in the experimental stage, sufficient equipment was not available for proper control and it was more difficult to secure the type of concrete desired. It should also be mentioned that the specifications for this project required a modulus of rupture of only 375 p.s.i. at seven days and only 450 p.s.i. at 28 days. The Texas Highway Department now considers such strengths for concrete bases to be definitely inadequate. Attention is directed to the small percentage of low cement factor concrete pavement in Texas classed lower than Grade B.

As stated in the opening paragraph of this paper, the objective of the Texas Highway Department in the design and construction of concrete pavement is to secure concrete of the proper quality for the required service with the maximum economy. The Department feels that the absolute volume method of design under the strength and quality specification guarantees concrete of the proper quality and the records of the service life of the pavements in use for eight years in this State are furnished as evidence to substantiate this statement. The Texas strength and quality specification designed in accordance with the methods outlined in this paper places a premium on the better materials, thereby providing concrete of the proper quality with the maximum economy.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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Discussion of a Paper by E. B. Cape:

Design and Control of Concrete Paving Mixtures —Texas*

BY F. H. JACKSON, THE AUTHOR AND OTHERS

CONVENTION DISCUSSION

F. H. Jackson†—Although I have no prepared talk, I would like to take, if I may, a few minutes to comment on Mr. Cape's very interesting paper. As he stated, the Texas Highway Department has used two types of specifications to govern concrete proportions for pavement work. In one, a minimum cement factor and a maximum water-cement ratio are specified, while in the other proportions are controlled through a so-called design strength or quality specification. The first type of specification has up to this time been required by the Public Roads Administration on all Federal-aid work, because it has felt that it was unwise to go beyond a certain limit in the reduction in cement content which seems to be possible because of the very high strengths being obtained in Texas. Up to the present time we have insisted that a minimum of 5 sacks of cement per cubic yard be used in concrete in pavements in which we participate to the extent of furnishing federal funds.

In 1939, year before last, the Commissioner of Public Roads, at the request of the Highway Department, instructed me to go to Texas and examine some of these low cement factor roads and, in company with representatives of the Highway Department, to determine their present condition. As Mr. Cape has said, certain of these pavements have been down now for upwards of 8 or 9 years. It would seem, therefore, that they had almost reached the point where they

*A. C. I. JOURNAL, Feb. 1941; *Proceedings* V. 37, p. 413.
†Senior Engineer of Tests, Public Roads Administration, Washington.

FIG. A—LIVE OAK COUNTY, TEXAS PAVEMENT BUILT 1933 (STATE PROJECT 931) CEMENT FACTOR, 3.1 SACKS PER CU. YD., PHOTOGRAPHED FEBRUARY 1934

FIG. B—SAME AS FIG. A, PHOTOGRAPHED MARCH 1939

could be evaluated in terms of service behavior. I have traveled over a number of these roads in company with Mr. Cape and other representatives of the Highway Department, and have examined them with considerable care. In most of Texas the climatic conditions, as you know, are such that we did not feel that durability as we consider

it in the northern part of the United States was a very important factor in determining the mix to be used. Freezing weather does not occur to any great extent in Texas, particularly in the southern part of the State, and for that reason we felt that it would not be necessary to impose the same requirement as regards maximum water-cement ratio as is necessary in the Northern States. We also knew from extensive test records that strengths meeting all of the usual design requirements were being obtained with very low cement factors. This may be due partially, as Mr. Cape has said, to the fact that they have very good aggregates in Texas. A partial reason may also lie in the very excellent field control exercised by this State. The problem may therefore be stated in the form of two questions; first, does the use of very low cement factors result in excessive surface wear or raveling and second, does the low cement content tend to produce such harsh and unworkable mixtures as to result in the formation of stone pockets, honeycomb areas, etc., in the surface. Our inspection was designed to ascertain the facts in regard to these two points.

Fig. A shows a concrete pavement in Live Oak County, south of San Antonio, in which the cement factor is 3.1 sacks of cement per cu. yd. The picture was taken in 1934, one year after the concrete road was constructed. This is one of the main routes to the lower Rio Grande Valley. The average traffic on this road is, I think, in the neighborhood of 1200 or 1500 vehicles per day, including a relatively large number of rather heavily loaded trucks, carrying fruit from the Rio Grande Valley to the north. Fig. B shows the same road as photographed in 1939, 6 years later. The difference in the surface texture is at once apparent. The very thin mortar surface which was formed on top of the pavement in the process of finishing has worn off, leaving the coarse aggregate exposed over a large proportion of the area. That is characteristic of these low cement factor roads which were built as Mr. Cape has said with the maximum amount of coarse aggregate that can be crowded into the mix and still maintain workability. The result is that they do not have a large excess of mortar floated to the top during the finishing operation. Instead, a very thin layer of mortar is formed which wears off within the first year or so, leaving the surface in the condition shown in Fig. B. Now a man from the North going down there and looking at these roads without previous experience, would immediately conclude that this condition represented the first stage of disintegration. This is not the case. A comparison of photographs of these roads taken year after year shows that they change very little after the surface mortar has been removed.

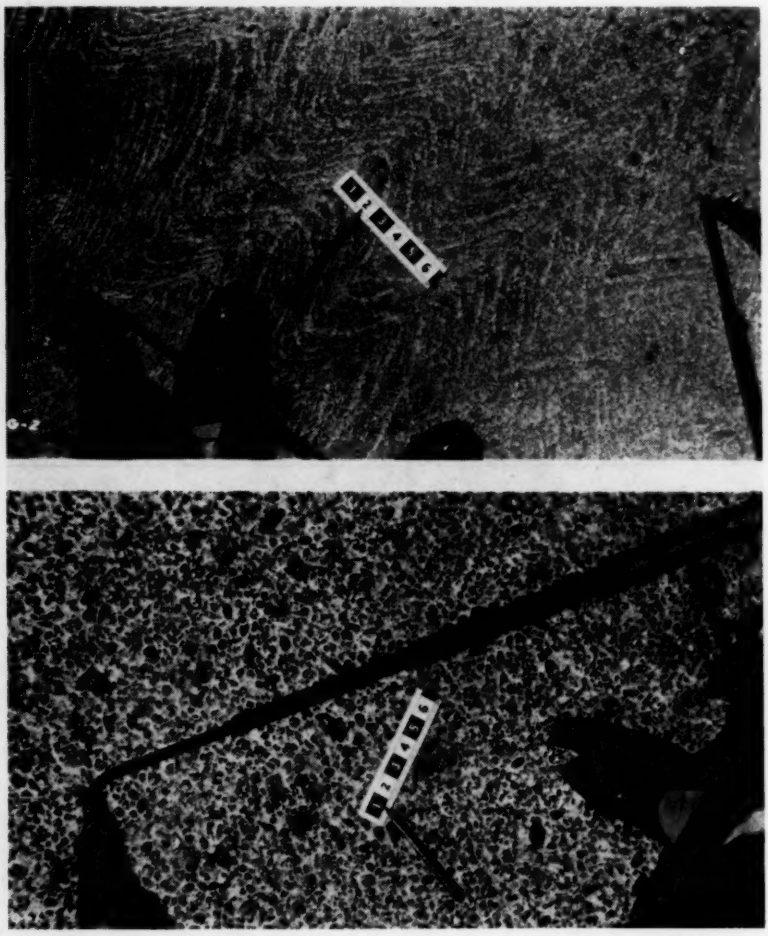

FIG. C—REFUGIO COUNTY, TEXAS (PROJECT 840A); BUILT 1932, CEMENT FACTOR, 3.7 SACKS PER CU. YD., DETAIL OF ORIGINAL FINISH

FIG. D—SAME AS FIG. C; PHOTOGRAPHED MARCH 1939, AFTER REMOVAL OF THIN MORTAR TOP BY WEAR

It is impossible, in many cases, to tell by inspection of the photographs in which year the picture was taken.

Fig. C is a close-up of the texture of the surface of one of these low cement factor roads as placed before the thin mortar top has worn off. The photograph shows the screed marks on the surface just as the road was constructed. Fig. D is a detail showing the same surface after the thin mortar top had been removed. This is a close-up of the texture

FIG. E—SAME ROAD AS FIG. C AND D—TYPICAL SURFACE AFTER 7 YEARS; PHOTOGRAPHED MARCH 1939

FIG. F—SHELBY COUNTY, TEXAS PAVEMENT (PROJECT 556) BUILT 1935; CEMENT FACTOR, 3.3 SACKS PER CU. YD., DETAIL SHOWING ABSENCE OF SPALLING AT TRANSVERSE JOINT, PHOTOGRAPHED MARCH 1939

and shows that the concrete contains a very hard flinty aggregate and the particles protrude very slightly from the surrounding mortar. However, the mortar is apparently of sufficient strength so that it is not worn out between the particles of aggregate. It has very good bond

and the surface is very smooth riding. This road was 7 years old when the photograph shown in Fig. D was taken. Cement factor, 3.7 sacks per cubic yard. Fig. E is a general view typical of the surface of which Fig. D is a detail.

Fig. F is a detail of a transverse joint on a job built in 1935 and containing 3.3 sacks of cement per cubic yard. The photograph taken in 1939 shows that there has been very little raveling at the joints in spite of the lean mix used. This particular joint was not any too well maintained from the standpoint of the seal. The filler is considerably below the surface but there appears to be practically no raveling. The edges are clear cut and the concrete is in excellent condition. Fig. G illustrates a condition which is rather interesting. This low cement factor concrete is, of course, not as hard and as wear-resistant as if it were richer. The wear that I speak of therefore may become a little more serious on sections where gravel can be tracked onto the pavement from adjacent roads, etc. This is one particular location where a gravel road comes into the concrete pavement and the traffic entering the concrete tracks gravel onto the road. Right at this point the wear seems to be a little more serious than over the pavement as a whole. This particular pavement has four sacks of cement per cubic yard of concrete and is carrying a traffic, I think, in the neighborhood of 5,000 vehicles a day. It is just north of Ft. Worth and is a very heavily traveled road, even in terms of some of our large cities in this part of the country. That pavement is in good condition. There is some slight evidence of the surface wear which I speak of, though not so marked as where the cement factor is considerably below four sacks.

And now a few comments on the second point; that is, workability. Fig. H illustrates the type of local defect which appears in the surface of those roads when the workability factor is not carefully controlled. Of course there is a limit to the amount of coarse aggregate which can be crowded into the mix. Experience has indicated that the maximum workability factor (b/b_o dry, loose basis) which can be used in this work is about 85. If this value is exceeded very much you are apt to get the condition shown, a local honeycomb spot. Here again, however, these potentially weak areas, even when they do occur, do not seem to be particularly serious under the conditions pertaining in Texas. The interesting feature in regard to this particular road is that the spot shown in Fig. H has been photographed five different times between 1933 and 1940, and if I take the five photographs and spread them out before you, you cannot tell one from the other. This illustrates again how little the concrete changes after the initial mortar surface has worn off. This, of course, does not mean that such a

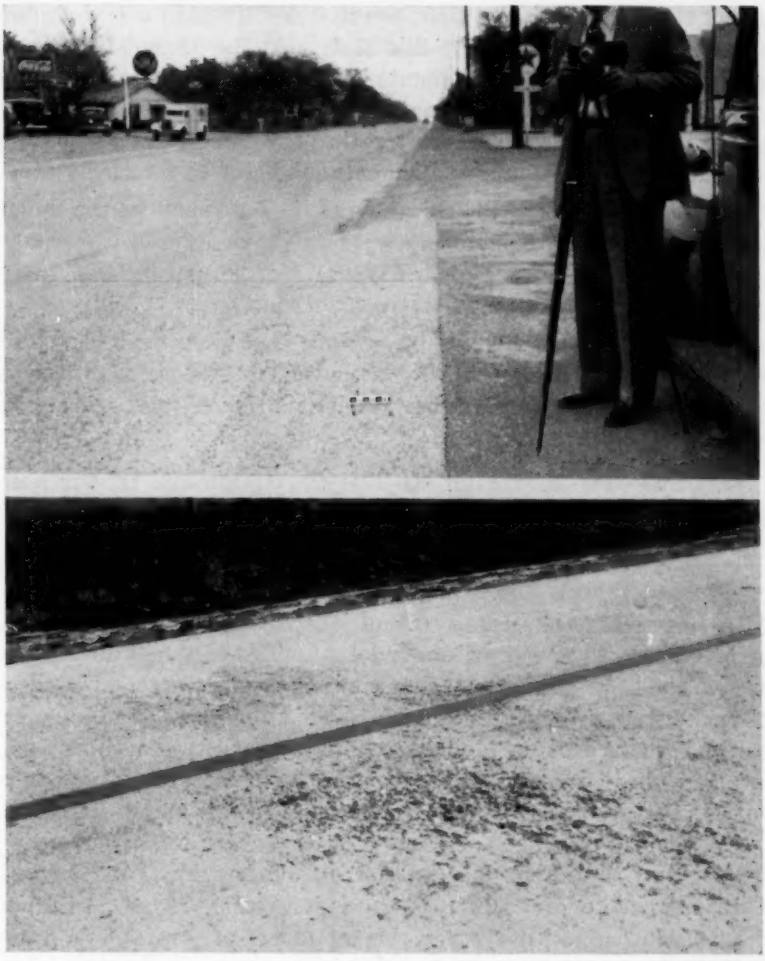

FIG. G—TARRANT COUNTY, TEXAS PAVEMENT (PROJECT 977B) BUILT 1932; CEMENT FACTOR; 4.1 SACKS PER CU. YD., DETAIL SHOWING SURFACE WEAR AT INTERSECTION WITH GRAVEL ROAD, PHOTOGRAPHED MARCH 1939

FIG. H—LIVE OAK COUNTY (PROJECT 931) 1933, CEMENT FACTOR 3.1 SACKS PER CU. YD., DETAIL OF LOCAL HONEYCOMB AREA, PHOTOGRAPHED MARCH 1940

condition should be encouraged and, as I have indicated, it can be practically eliminated by maintaining the workability factor below 85.

In general, our study of these low cement factor roads has convinced us that, under the conditions pertaining in Texas, cement

factors considerably lower than the national average may be allowed in concrete pavement work in this State. At the present time, specifications for concrete for pavements in Texas approved for Federal-aid are strictly on a flexural strength basis, with a specified upper limit on the permissible water-cement ratio of 8 gal. per sack for normal work and 7 gal. per sack for pavements subjected to very heavy traffic.

*G. A. Grant**—Would the thickness of the pavement be the same for different cement contents?

Mr. Cape—We have followed the standard nine-six-nine sections almost exclusively. For several years afterward, up until 1935, we had the four foot thickened edge. Since that time we have had the two foot thickened edge. We are now going to an eight-six-eight section, two parabolic curve; this design is used on some of the roads.

J. H. Chubb†—What is the maximum size of the aggregate?

Mr. Cape—The maximum sized aggregate allowed in our specifications is three inches. However the maximum aggregate found in 95 per cent of our commercial sources is around two inches.

A. S. Butterworth‡—What is the compressive strength?

Mr. Cape—Let me refer to several jobs; I will read these notes over hurriedly. On the Shelby County job of which Mr. Jackson showed a picture, the core strength at the time that job was completed was 3971 psi.; average core strength now, 5306, an increase of 33.6 per cent. A second Shelby County job had 3300 average core strength originally; now 4982, an increase of 51.3 per cent. In Nueces County that had two sacks in it originally, and which we are still using, had a 1780 psi. average core strength originally; but now it is 3550, an increase of 99.4 per cent. Another job in Nueces County which contained a 3.6 sack cement factor was 4307 psi. and is now 5265. A job in Cameron County, which had an average cement factor of from 2.8 to 3.2 originally had 3342 psi.; now 5240. That is representative of the majority of our jobs. A four sack job in Potter and Carson County originally had 4715; now has 5710 core strength. The ratio of our core strength to beam strength is around eight to one, and for 600 lb. beam we expect to produce 4800- or 5000-lb. concrete. With 85 per cent of our aggregates, we can get a 600 lb. beam strength with around 3.7 sacks.

*Robert B. Gage***—I noticed in one picture shown that there was not travel enough on the outside road to wear the grass away. If there is not enough travel to wear the grass down, it is going to last. In

*Canada Cement Company, Montreal.
†Penn-Dixie Cement Corp., New York City.
‡General Materials Co., St. Louis, Mo.
**New Jersey State Highway Dept., Trenton.

those roads the grass ran right up to the pavement and there was no sign of a shoulder and no evidence that it wore any of it away.

A Member—I would like to ask Mr. Cape if these roads are subject to frost action?

Mr. Cape—The Carson County job is just out of Amarillo; I believe the average cycles of freezing and thawing on that job are 54 a year. The average job in Texas, though, is not subject to that many cycles. We have had no trouble of the type that has been experienced in the north. I would like to call one other thing to your attention, and that is the low water content measured in gallons per cubic yard. Our pavement has from 28½ to 31 gal. of water per cu. yd. of concrete, and as to our aggregate, I want to call another thing to your attention, that is that our aggregates are from commercial sources. In Texas the specific gravity varies from about 2.2 to 3.1; therefore you cannot gauge the density of our concrete by the unit weight.

*R. B. McMinn**—I would like to ask Mr. Cape if that is a non-skid surface?

Mr. Cape—Yes, it is a wonderful surface, more non-skid than the original herringbone finish that you would have otherwise.

Mr. Gage—May I ask what the aggregate is?

Mr. Cape—We have every type of aggregate you can think of in Texas.

A member—What are the intensities of those freezing and thawing cycles? What low do you get in winter and how high do you get in summer?

Mr. Cape—Our average lows in Texas are around plus 20 F. However, at Amarillo, which is subjected to so many cycles, the low there is minus five or minus ten; that is the Potter-Carson County job.

Mr. Jackson—Isn't it true that in the Panhandle, where you have low temperatures, they occur at a season of the year when there is not very great precipitation?

Mr. Cape—That is correct.

Mr. Jackson—You do not have the same combination of low temperature and moisture that we have in the north, and that makes, in my opinion, a lot of difference. I do not believe they could get away with this low cement content in the north for one minute—in Minnesota or Wisconsin for instance.

Mr. Butterworth—Was any allowance made for tire wear?

Mr. Cape—Mr. Jackson questioned the quality of the concrete in the completed road. In 1939, while he was down there, we went out

*Washington, D. C.

on the roads to points that he selected. Mr. Gillette, the Public Roads Administration representative, went with me and we cut large slabs, 6 x 6 ft., out of the pavement with our core machine; took them into the laboratory and with a large carborundum saw, sawed the slabs into beams, and shipped Mr. Jackson a whole carload of beams and cores and I imagine he still has part of the specimens in the laboratory. He tested the surfaces of the slabs, by means of a shot device to gage their resistance to wear, and I think that he came to the conclusion that they would resist all of the wear that they were subjected to.

*Guy H. Larson**—Wisconsin has some pavements up there with three and a half sacks of cement to the yard, which have been in use now over ten years and the pavement seems still to be in a very good condition.

Arthur A. Levison†—What has been your observation of the cracking of the low ranges of your low cement content pavements?

Mr. Cape—We have found no difference in the cracking of our low cement pavements and the cracking on the higher cement factor pavements. It depends entirely upon the subgrade conditions. Where we have a good subgrade material, there is practically no cracking on the low cement factor jobs and practically none on the high cement factor jobs. Where we have our black farm land and very poor subgrade conditions, regardless of the cement factor, the pavement is cracked.

*Wisconsin State Highway Commission, Madison, Wisc.
†Blaw Knox Co., Pittsburgh, Pa.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

Long-Time Overload Tests of a T-Beam Floor Panel*

By L. E. Grinter†
MEMBER AMERICAN CONCRETE INSTITUTE
and Burge Kepford‡

SYNOPSIS

A simple T-Beam floor panel designed for light live loading was tested under the action of concentrated loads and under the uniform A.C.I. Code loading. The panel was then overloaded for six months by the application of double the A. C. I. Code loading. Finally, after time for recovery, the original test loadings were replaced and their actions are compared with the original strains and deflections. The ability of the slab to carry 3.7 times the design loading and then to act elastically under normal loads is established.

In the fall of 1937 tests were started on a cast-in-place T-beam floor panel at Armour Institute of Technology.** The initial tests were reported in a Technical Bulletin published by the Institute as of August, 1938. The original tests have now been supplemented by a semi-destructive long-time period of loading followed by a repetition of the original load tests. Thus, data have been obtained upon the ability of the panel to sustain severe overload for a long time and upon the ability of a severely over-stressed panel again to act elastically under normal loadings. This report will cover the action of the panel illustrated by Fig. 1. The data were accumulated in successive years by Fenton Harding, Carl Shermer, and Burge Kepford, graduate assistants assigned to this investigation.

The test panel—The panel shown in Fig. 1 was constructed to represent standard practice in so far as it could be reproduced to about one-half the usual scale. Pea gravel concrete was used. The

*Received by the Institute Oct. 14, 1940.
†Vice President and Dean of the Graduate School, Illinois Institute of Technology.
‡U. S. Bureau of Reclamation, Green River, Wisc.
**Now Illinois Institute of Technology.

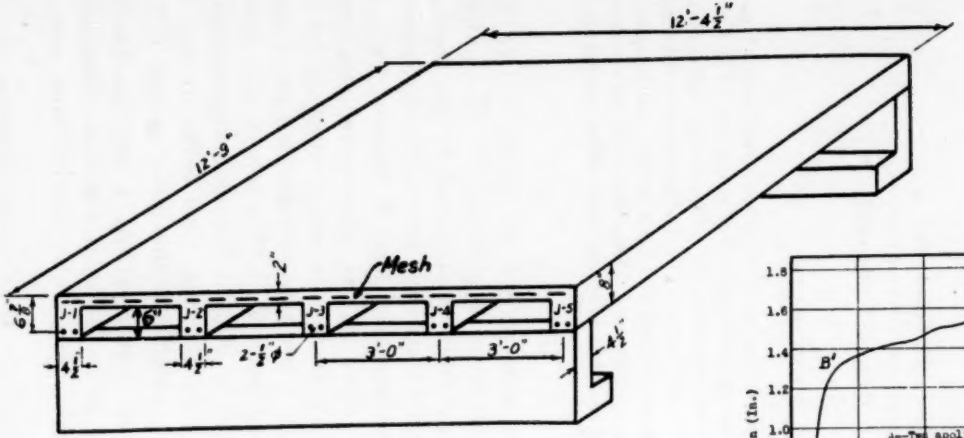

FIG. 1—THE T-BEAM FLOOR PANEL WHICH WAS TESTED

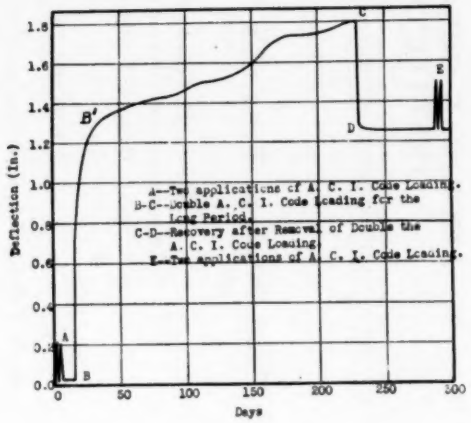

FIG. 2 (RIGHT)—CUMULATIVE DEFLECTION FOR CENTER JOIST OF T-BEAM PANEL UNDER A. C. I. CODE LOADING

average compressive strength of the concrete at 54 days was found to be 5860 p.s.i., the secant modulus of elasticity being 4,150,000 p.s.i. The steel bars showed an elastic limit of 44,500 p.s.i. and an ultimate strength of 78,000 p.s.i. Elongation was 25 per cent before failure. The panel was designed for a live load of 50 lb. per sq. ft. Its dead load was calculated to be 35 lb. per sq. ft. Hence the A. C. I. Code loading of one and one half times the live load plus one half of the dead load was 92.5 lb. per sq. ft.

Test loadings—The sequence of loadings under which the action of the panel was studied were as follows:

(1) Concentrated loads of 2000, 3000, and 4000 lb. applied at the mid-span of the center joist.

(2) A. C. I. Code loading which was a superimposed uniform load of 92.5 lb. per sq. ft. equivalent to one and one-half times the design live load plus one-half of the dead load.

(3) Repetition of the A. C. I. Code loading 24 hours after its removal.

(4) Double the A. C. I. Code loading left in place for six months during which severe cracking occurred.

(5) A. C. I. Code loading replaced upon the severely cracked panel.

(6) Repetition of the A. C. I. Code loading 24 hours after its removal.

(7) Repetition of the concentrated loading tests.

It was felt that these tests were adequate to determine how seriously the ability of the panel to perform its normal function had been influenced by severe long-time overload.

Cumulative deflection—The illustration, Fig. 2, shows the cumulative deflections of the center joists during a period of 300 days. Deflections produced by the concentrated loads are not shown on Fig. 2 because they produced no appreciable permanent deflection. The severity of the long period loading test is evident from the fact that it caused a permanent deflection of over 1.2 in. Cracks into which the point of a lead pencil could be inserted opened up in the concrete joists during this period.

Cumulative steel strain—Corresponding to the upper part of the deflection curve ($B'C$) from Fig. 2, there is reproduced in Fig. 3 a curve illustrating the gradual increase of steel strain. The increase of strain during this period amounted to 0.002 in. per in. If the action had been elastic, the corresponding increment of steel stress during this period of 180 days would have been 60,000 p.s.i. Actually, of course, this strain in the steel represents flow after the initial elastic

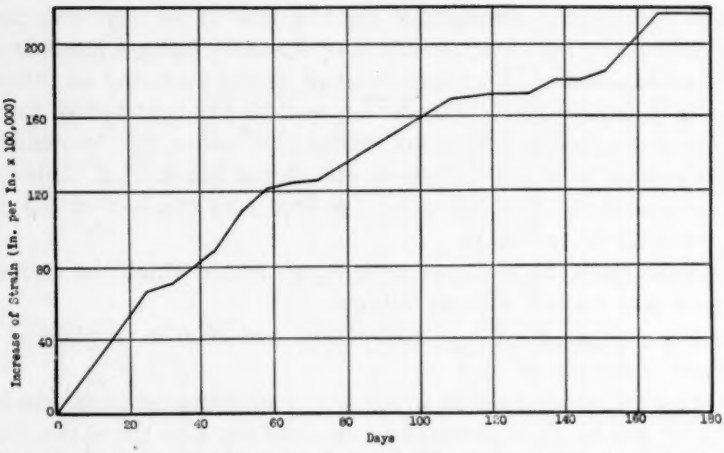

FIG. 3—INCREASE OF STEEL STRAIN IN CENTER JOIST OF T-BEAM PANEL UNDER DOUBLE THE A. C. I. CODE LOADING FOR 180 DAYS

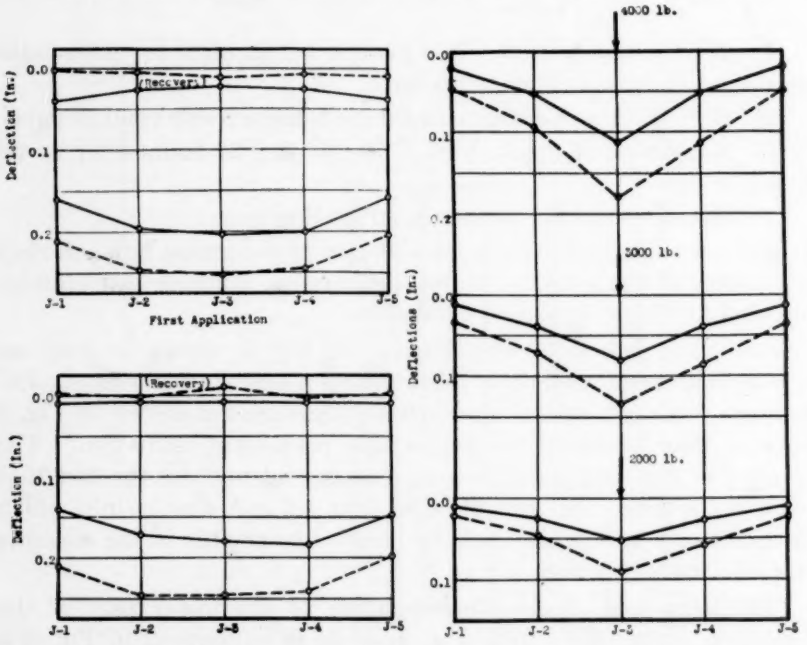

FIG. 4 (LEFT)—DEFLECTIONS UNDER A. C. I. CODE LOADING FOR T-BEAM PANEL; AT TOP, FIRST APPLICATION; (BELOW), SECOND APPLICATION 24 HOURS LATER

FIG. 5 (RIGHT)—DEFLECTIONS UNDER CONCENTRATED LOADING FOR T-BEAM PANEL

(Solid lines, before overstressing; dash lines, after overstressing)

and plastic steel deformation corresponding to the part BB' of the deflection curve of Fig. 2 had already taken place. During this period of plastic flow, some of the gage holes on the steel rods actually became elliptical in shape and these gage lengths had to be abandoned. Fortunately, there were six gage lengths prepared on the rods of each joist so that satisfactory strain data could still be obtained even after about one-fourth of the gage lengths had been abandoned.

Action before and after overstressing—There are reproduced in Fig. 4 several deflection curves showing the influence of the A. C. I. Code loading upon the panel before and after it had been severely overstressed. The recovery is also indicated. The broken lines are for the panel after it had been severely damaged by overstress. It is noted that the deflections under A. C. I. Code loading have been increased by the long period of overstress but that these deflections are even more nearly elastic than the deflections caused by the original loading. This is indicated by the fact that recovery is almost 100 per cent of the deflection. That recovery of the joist J-3 was found to be more than 100 per cent in one instance was attributed to slip in the connections of the deflection gage.

Influence of concentrated loads—In Fig. 5 the deflections of the five joists are illustrated for the effects of three load concentrations. The load was placed at the mid-point of the center joist J-3. Again it is evident that the overstressed panel has much heavier deflections than the panel before it was loaded beyond its elastic limit. However, recovery under concentrated loading was still 100 per cent so that it is reasonable to assume that the panel could still withstand an indefinite number of repetitions of the concentrated load of 4000 lb. which approached its calculated capacity loading. This point is emphasized in Fig. 6 where it is shown that the steel strain under the 4000-lb. concentrated load increased from an equivalent stress of 11,000 p.s.i. to one of 17,000 p.s.i. by the influence of the long-period overloading. Nevertheless, recovery was still found to be complete.

Conclusions—These tests seemed to the authors to establish rather conclusively the following points.

(1) A reinforced concrete floor panel of T-beam construction has a great reserve capacity of strength. This is shown by the fact that the test panel carried for a period of six months a superimposed uniform loading of 185 lb. per sq. ft. which was 3.7 times the design live loading. During this period, the joists cracked severely and deflections increased gradually, but there was no reason to feel that the panel was approaching immediate failure when the load was removed.

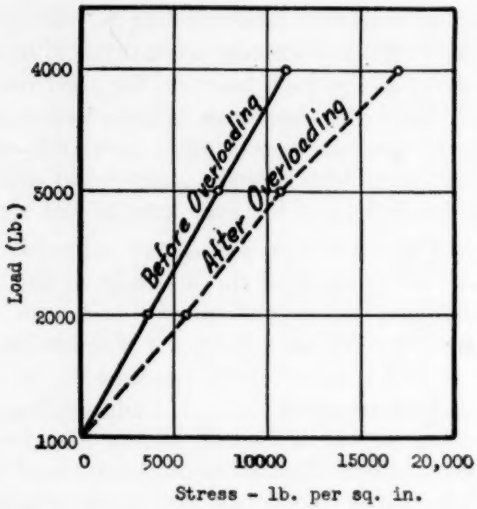

FIG. 6—INCREASE OF STEEL STRAIN FOR CENTER JOIST OF T-BEAM PANEL UNDER CONCENTRATED LOADING AT 4000 LB.

(2) After the panel had been overloaded in this manner, it was retested under A. C. I. Code loading (superimposed uniform load equal to one and one-half times the live load plus one-half of the dead load) and also under normal concentrated loadings. In each case the panel showed deflections and steel strains that were considerably heavier than those observed originally but these deflections and strains were found to be elastic. Hence, it is concluded that these loadings might have been repeated almost indefinitely without producing further serious distress.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

Concrete Exposed to Sulphur Water*

By John S. Nelles†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This is a progress report on The Detroit Edison Co. tests showing the condition of concrete specimens comprising different mixes, admixtures, and cements; after 12 years exposure, completely submerged, in flowing sulphur water. The data include compressive strength curves and extent of disintegration, from which some conclusions have been drawn and some recommendations made.

In 1927, work was started on foundations for new Delray Power Plant No. 3 in Detroit, Michigan. The building columns and generator bases were to be supported by concrete caissons carried to bedrock, approximately 94 ft. below grade. Open holes were excavated and lined with pre-fabricated caisson cylinders made from 3-in. tongue and groove yellow pine, reinforced with steel plate splice rings at joints between cylinder sections. When sulphur water and gas were encountered coming through the rock seams, it was decided to grout the seams before proceeding with the work. Cased holes were drilled throughout the foundation area, down into the rock, and portland cement grout was forced into the seams under 110-lb. pressure. A grout, composed of equal parts of cement and Celite or Pumicite, was also used. In this manner the caisson bottoms were successfully sealed against sulphur gas or water.

In order that definite information might be obtained as to the effect of this sulphur water exposure on the types of concrete actually used, it was decided to conduct a test on the job. Later many other types of concrete mixtures, admixtures, and special cements were added to the test.

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†Engineer, The Detroit Edison Co., Detroit.

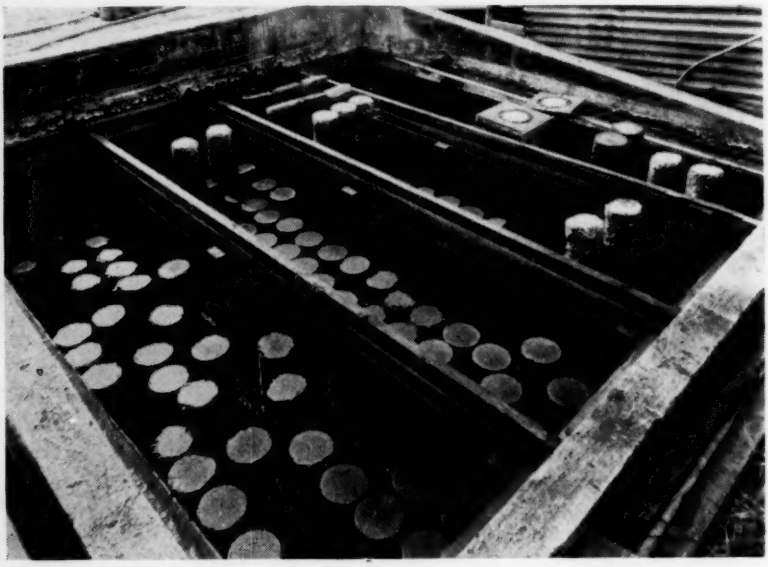


FIG. 1—HALF OF TEST PIT SHOWING BAFFLES WHICH CONTROL THE FLOW OF SULPHUR WATER THROUGH THE PIT

TEST PROCEDURE

Two concrete pits, 11 ft. square and approximately 1 ft. 8 in. deep, were built at grade. (Fig. 1.) A 12-in. cased hole was drilled to rock and lined with 6-in. i.d. wooden pipe, grouted in place. Sulphur water was encountered under sufficient pressure to force a continuous flow up into the tank, except during mid-summer when it is necessary to pump. Fresh water was piped into the other tank and overflow pipes were installed to maintain uniform level and to carry waste water to canal. Analyses of sulphur water are given in Table 1. Once a year the tanks are flushed out to get rid of sediment, and during the winter months ample protection against frost is provided.

The specimens are 6x12-in. compression cylinders and, in general, have been completely submerged in the tanks. It is realized that this exposure is not as severe as the partial submersion, but it more nearly approximates the job conditions for the reason that all caissons are well below grade. At later periods a few slabs were made and exposed, half-submerged, along with a very few of the original 6x12-in. cylinders.

The breaking schedule was arranged to give precedence to the ten-year and older ages, and consequently the few specimens broken

at earlier ages are insufficient to give good average values, but sufficient perhaps to suggest a trend.

TABLE 1—ANALYSES OF SULPHUR WATER

Ionic Values	Parts per Million by Weight				
Item	1928	1929	1930	1931	1940
Calcium ($Ca++$)	528		558	540	562
Magnesium ($Mg++$)	36		161	194	223
Sodium ($Na+$)	109		345	510	714
Bicarbonate (HCO_3-)	346		412	430	413
Sulphate (SO_4--)	1580		1540	1625	1716
Chloride ($Cl-$)	1296		530	730	1240
Sulphide ($HS-$)	39	90	76	101	119
Calcium Sulphate		2275			
Probable Combinations					
Sodium Chloride ($NaCl$)	2130		743	1115	1816
Sodium Sulphate (Na_2SO_4)	662		....	....	*....
Sodium Hydrosulphide ($NaHS$)	67		129	171	....
Calcium Bicarbonate ($Ca(HCO_3)_2$)	467		548	572	548
Calcium Sulphate ($CaSO_4$)	1402		1427	1345	1448
Magnesium Sulphate ($MgSO_4$)	178		661	828	868
Magnesium Chloride ($MgCl_2$)			107	99	186

*Since 1928 added experience has proved that the probable amount of sodium sulphate cannot be even closely approximated.

MIX DESIGN FOR JOB CONCRETE

As noted in a paper presented at the A. C. I. 1929 Convention,* many engineers who have studied similar problems were consulted, and little definite information was obtained as to the type of mix to use. There did seem to be evidence favorable to a so-called Super cement, and it was finally decided to use it in conjunction with a rich mix, in the bottom half of the caissons, changing over to a leaner mix, with standard portland cement for the top half. This was done for economy, because it was believed that the successful sealing off of the injurious sulphur water was the real protection.

At that time we were using the fineness modulus method of mix design, with a water-cement ratio corresponding to the equation, $S = \frac{14{,}000}{8x}$. Design data for the various mixes are shown in Table 2.

DISCUSSION

Fig. 2 shows compressive strengths of the caisson concretes under various curing conditions, and Fig. 3, the appearance of the specimens after 12½ years exposure. It is hard to explain the consistently lower strengths at 12½ years, because the cylinders cured in fresh water and damp sand, also broke lower, and these latter showed no signs of disintegration whatever. We will have to wait for the next

*"Three and a Half Years Experience of The Detroit Edison Co., in Concrete Control," A. C. I. *Proceedings* Vol. 25, p. 100.

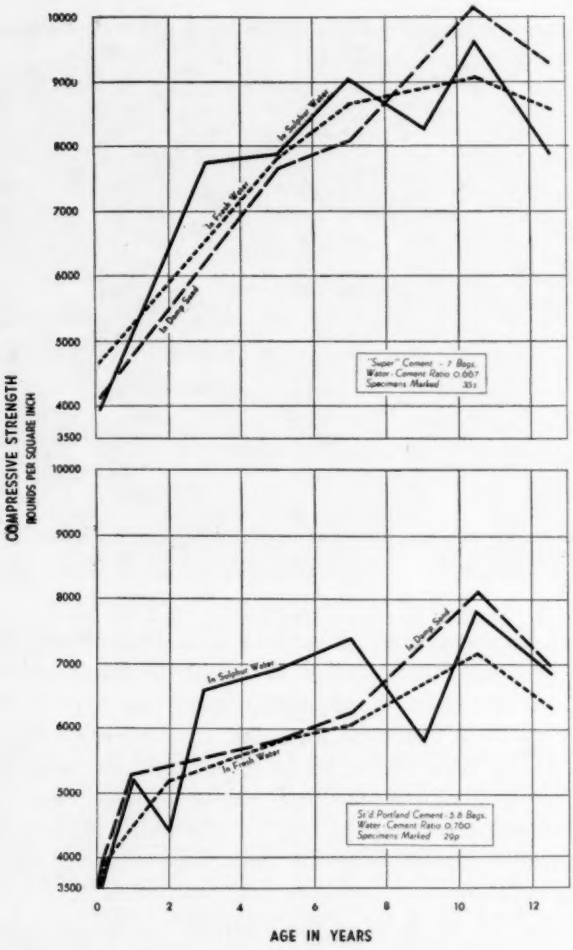


FIG. 2—COMPRESSION STRENGTHS OF CAISSON CONCRETE UNDER DIFFERENT CURING CONDITIONS

series of breaks, possibly three years hence, to determine just what is happening.

Fig. 4 includes comparative strength curves for Super and standard portland cements. A study of these curves, together with those in Fig. 3, would indicate some advantage in Super cement, especially in the richer mix.

Fig. 4 also gives compressive strengths for Lumite and Incor cements, and Fig. 8 shows extent of disintegration after 11 years exposure. The Incor does not seem to show any advantage over standard port-

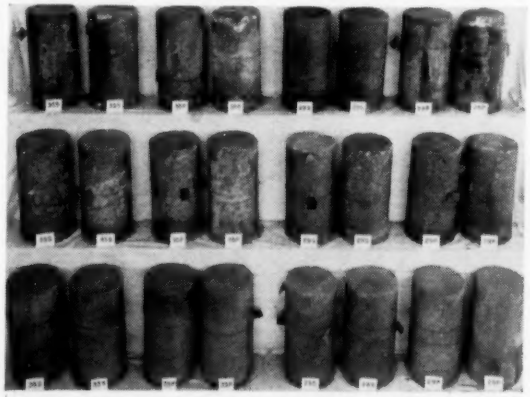


FIG. 3—AFTER 12½ YEARS EXPOSURE SUBMERGED—IN SULPHUR WATER, TOP ROW; IN FRESH WATER, CENTER ROW; IN DAMP SAND, BOTTOM ROW

Specimen Numbers—Top row: 35S, 35S, 35P, 35P, 29S, 29S, 29P, 29P

Center Row: 35S, 35S, 35P, 35P, 29S, 29S, 29P, 29P

Bottom Row: 35S, 35S, 35P, 35P, 29S, 29S, 29P, 29P

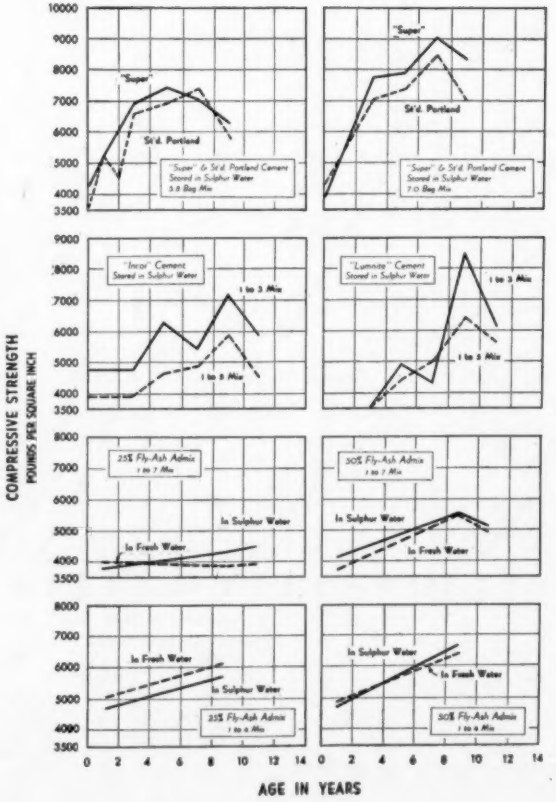

COMPRESSIVE STRENGTH
POUNDS PER SQUARE INCH
"Super"
Std. Portland
"Super" & Std. Portland Cement Stored in Sulphur Water 5.5 Bag Mix
"Super" & Std. Portland Cement Stored in Sulphur Water 7.0 Bag Mix
"Incor" Cement Stored in Sulphur Water
"Lumnite" Cement Stored in Sulphur Water
1 to 3 Mix
1 to 5 Mix
25% Fly-Ash Admix 1 to 7 Mix
50% Fly-Ash Admix 1 to 7 Mix
In Sulphur Water
In Fresh Water
25% Fly-Ash Admix 1 to 6 Mix
50% Fly-Ash Admix 1 to 6 Mix
AGE IN YEARS

FIG. 4 (RIGHT)—COMPRESSIVE STRENGTHS OF EXPERIMENTAL CONCRETE

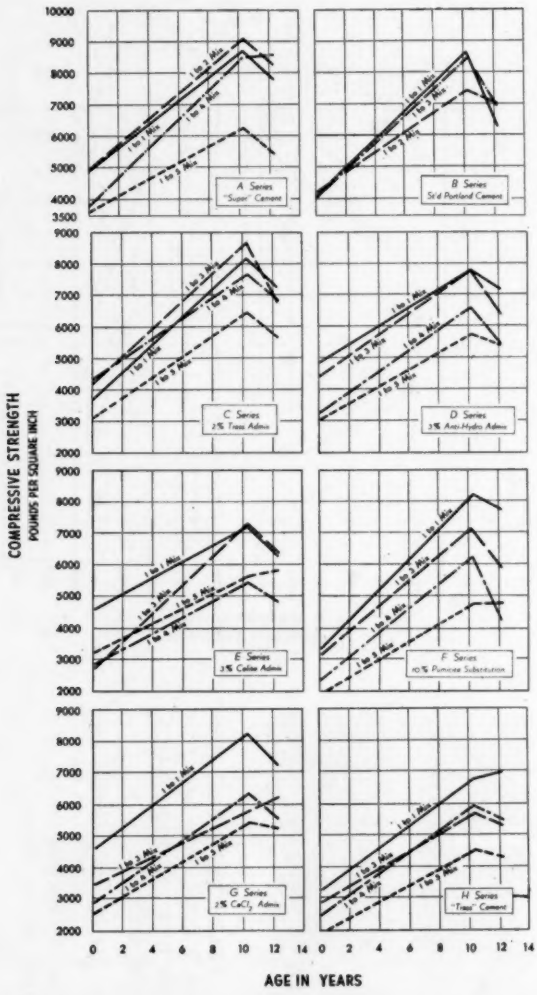


FIG. 5—COMPRESSIVE STRENGTHS OF EXPERIMENTAL CONCRETES STORED IN SULPHUR WATER

land cement, but the Lumnite, which is a high alumina cement, has come through the eleven years in excellent shape.

In Fig. 5, compressive strength curves are plotted for concrete mixes, Series "A" to "H," while Fig. 6, 7, and 8, show the extent of disintegration after 12¼ years submersion in sulphur water. The outstanding specimens of this group are the "H" Series, in which trass cement was used. These are in excellent shape down to the

1 to 4 mix. Even the 1 to 5 mix has stood up well, only just beginning to show signs of trouble. It will be noted that the "C" Series, with 2 per cent trass admix, (a volcanic ash), has not stood up as well as the trass cement. The latter is a portland cement with a percentage of trass intimately mixed. It is probable that there is more trass in the trass cement than the 2 per cent in Series "C," although we were unable to find out just what the proportion actually was.

The "F" Series has also come through in good shape, in fact, just a degree poorer than the "H" Series. It is interesting to note that here, too, a fifth material has been incorporated, in this case, Pumicite, also of volcanic ash origin.

Next in order of merit is the "A" Series Super cement, which was a portland cement into which a small amount of tannic acid was incorporated.

For the balance of the specimens there is little to choose between; the standard portland cement, "B" Series, showing up about as well or better than Series "D," "E," or "G."

For the last few years we have been doing a lot of work, using fly-ash in concrete, and the value of the fly-ash has been so apparent that we included two sets of specimens in this sulphur water test. This fly-ash is obtained from our Trenton Channel Power Plant, where pulverized coal is burned.

The first of these was the 4AT and 7AT Series, shown in Table 2, with strength curves in Fig. 4, and lack of disintegration in Fig. 9. These have had a 10¾-year exposure, and are in excellent shape. The strengths, too, are well up, especially when the leanness of the mix is considered. Even the leanest 1 to 7 mix rates the equal of the Lumnite or "H" Series.

The other fly-ash series comprises twelve 8x12-in. slabs 2-in. thick, with four reinforcing bars of ½-in. diameter embedded in the center of each slab—two slabs each of pebble-sand concrete, pebble-fly-ash concrete, and cinder-flyash concrete, in two mixes, 4-bag and 5-bag cement content. Mix data are shown in Table 3.

Specimens were put into the sulphur water tank at the age of 28 days, one slab of each type of concrete submerged, and the duplicate half-submerged. Fig. 9 shows the condition of these slabs after an exposure of 4¾ years.

Fig. 10 shows condition of cylinders that have been subjected to

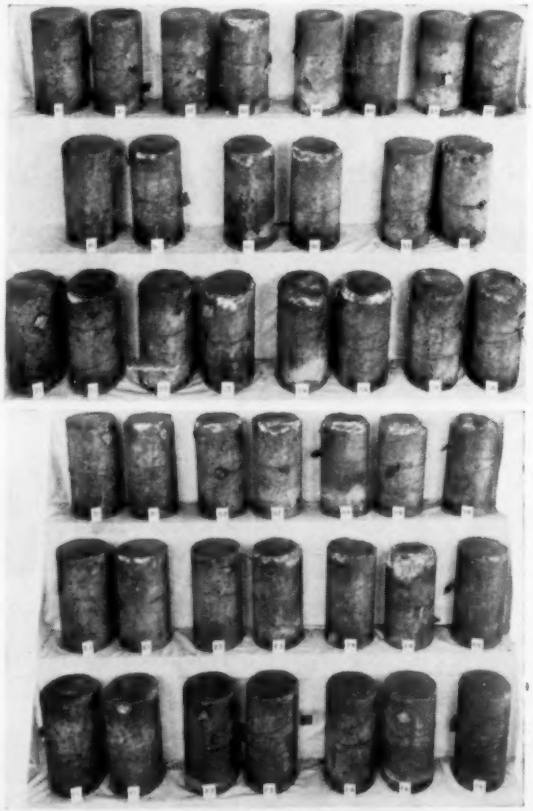
TABLE 2—DATA ON SPECIMENS UNDER TEST

Date	Type of Cement	Admixture % by Wt. of Cement	No. of 6" Dia. x 12" Speci-mens	Spec. Mkd.	Placed in Test Pits		Real Mix	Water Cement Ratio Gals. / Bag	Slump Ins.
					Age Days	Submerged or ½ Submerged			
Job Concrete									
1–16–28	Std. Portland	None	84	29P	2	Submerg.	1–4.40	5.70	4
1–24–28	"Super"	None	84	35S	2	Submerg.	1–3.45	5.00	4
Experimental Concrete									
1–24–28	Std. Portland	None	42	35P	2	Submerg.	1–3.45	5.00	3
1–30–28	"Super"	None	42	29S	2	Submerg.	1–4.40	5.70	3½
6–16–28	"Super"	None	12	A1	2	Submerg.	1–1	3.25	3½
6–15–28	"Super"	None	12	A2	2	Submerg.	1–2	4.00	3½
6–15–28	"Super"	None	12	A4	2	Submerg.	1–4	5.43	4½
6–15–28	"Super"	None	12	A5	2	Submerg.	1–5	6.15	4½
6–19–28	Std. Portland	None	12	B1	2	Submerg.	1–1	3.25	6
6–18–28	Std. Portland	None	12	B2	2	Submerg.	1–2	4.00	3¾
6–18–28	Std. Portland	None	12	B3	2	Submerg.	1–3	4.75	3½
6–20–28	Std. Portland	2% Trass	12	C1	2	Submerg.	1–1	3.25	4
6–19–28	Std. Portland	2% Trass	12	C3	2	Submerg.	1–3	4.75	4¼
6–19–28	Std. Portland	2% Trass	12	C4	2	Submerg.	1–4	5.43	3¾
6–20–28	Std. Portland	2% Trass	12	C5	2	Submerg.	1–5	6.15	3
6–19–28	Std. Portland	3% Anti-Hydro	12	D1	2	Submerg.	1–1	3.25	4½
6–19–28	Std. Portland	3% Anti-Hydro	12	D3	2	Submerg.	1–3	4.75	3¾
6–18–28	Std. Portland	3% Anti-Hydro	12	D4	2	Submerg.	1–4	5.43	3½
6–18–28	Std. Portland	3% Anti-Hydro	12	D6	2	Submerg.	1–5	6.15	4
6–20–28	Std. Portland	3% Celite	12	E1	2	Submerg.	1–1	3.25	4
6–20–28	Std. Portland	3% Celite	12	E3	2	Submerg.	1–3	4.75	3¾
6–20–28	Std. Portland	3% Celite	12	E4	2	Submerg.	1–4	5.43	6
6–20–28	Std. Portland	3% Celite	12	E6	2	Submerg.	1–5	7.15	3
6–21–28	Std. Portland	*10% Pumicite	12	F1	2	Submerg.	1–1	3.25	4½
6–21–28	Std. Portland	*10% Pumicite	12	F3	2	Submerg.	1–3	4.75	5½
6–22–28	Std. Portland	*10% Pumicite	12	F4	2	Submerg.	1–4	5.43	3
6–22–28	Std. Portland	*10% Pumicite	12	F6	2	Submerg.	1–5	6.15	3½
6–22–28	Std. Portland	2% Ca Cl	12	G1	2	Submerg.	1–1	3.25	3½
6–22–28	Std. Portland	2% Ca Cl	12	G3	2	Submerg.	1–3	4.75	4½
6–25–28	Std. Portland	2% Ca Cl	12	G4	2	Submerg.	1–4	5.43	4
6–25–28	Std. Portland	2% Ca Cl	12	G6	2	Submerg.	1–5	6.15	3½
6–26–28	"Trass"	None	12	H1	2	Submerg.	1–1	3.65	5
6–26–28	"Trass"	None	12	H3	2	Submerg.	1–3	4.75	5
6–26–28	"Trass"	None	12	H4	2	Submerg.	1–4	5.43	3¼
6–25–28	"Trass"	None	12	H6	2	Submerg.	1–5	6.30	3
10–1–29	"Incor"	None	28	IN3	2	Submerg.	1–3	4.50	3 to 4
10–4–29	"Incor"	None	28	IN5	2	Submerg.	1–5	6.40	3 to 4
10–2–29	"Lumnite"	None	28	LU3	2	Submerg.	1–3	4.50	3 to 4
10–3–29	"Lumnite"	None	28	LU5	2	Submerg.	1–5	6.40	3 to 4
1–8–30	Std. Portland	25% Fly-Ash	12	4AT25	8	Submerg.	1–4	6.68	6
1–6–30	Std. Portland	25% Fly-Ash	12	7AT25	8	Submerg.	1–7	8.55	5
1–9–30	Std. Portland	50% Fly-Ash	12	4AT50	8	Submerg.	1–4	7.55	7
1–7–30	Std. Portland	50% Fly-Ash	12	7AT50	8	Submerg.	1–7	9.50	5
10–15–35	Std. Portland	None	2	**	18	½ Subm.	1–5.4	6.93	1½
10–15–35	Std. Portland	None	2	***	18	½ Subm.	1–5.4	6.93	1½
12–20–35	Std. Portland	Fly-ash substituted for sand	4	6PPA	123	½ Subm.	6¼ Bags Cement	9.70	3

*Substitution for cement.

**Cast in wood molds and placed in sulphur water with molds left on.

***Cast in iron molds and placed in sulphur water with molds left on.



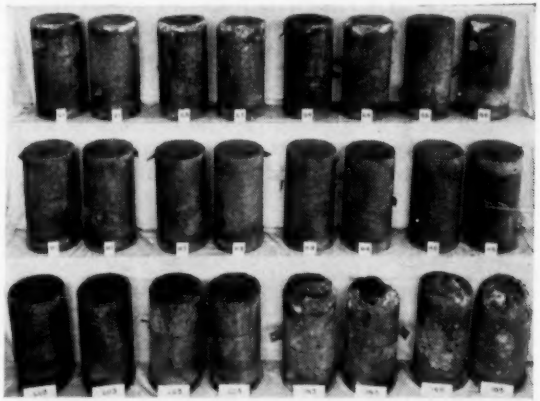
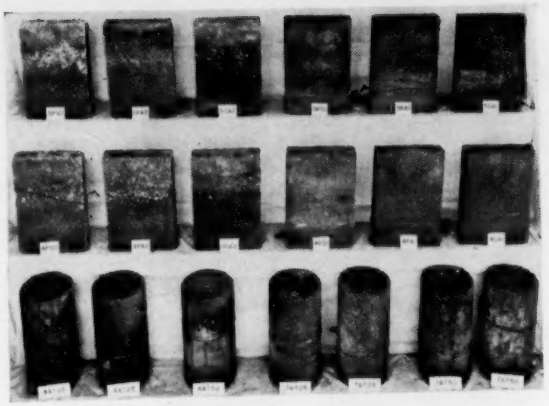


FIG. 6 (TOP LEFT)—SERIES "A" (TOP ROW), "B" (CENTER), AND "C" (BOTTOM) AFTER 12¼ YEARS EXPOSURE SUBMERGED IN SULPHUR WATER

FIG. 7 (BOTTOM LEFT)—SERIES "D" (TOP), "E" (CENTER), AND "F" (BOTTOM) AFTER 12¼ YEARS EXPOSURE SUBMERGED IN SULPHUR WATER

FIG. 8 (ABOVE)—SERIES "G" (TOP) AND "H" (CENTER) AFTER 12¼ YEARS EXPOSURE—LUMNITE (4 SPECIMENS AT LEFT BOTTOM ROW) AND INCOR (4 SPECIMENS AT RIGHT, BOTTOM ROW) SERIES AFTER 11 YEARS EXPOSURE—ALL SUBMERGED IN SULPHUR WATER



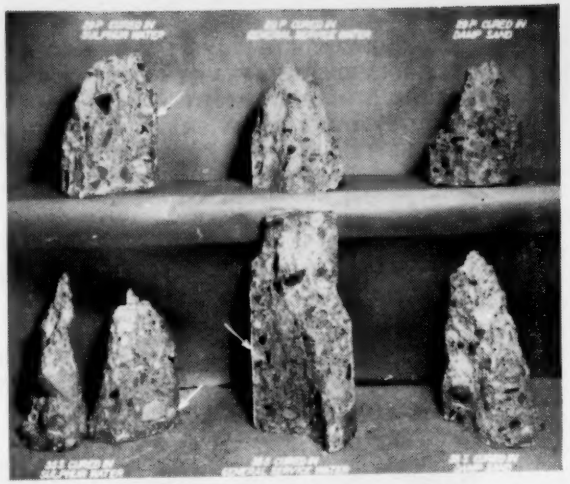


FIG. 9 (TOP LEFT)—PEBBLE-SAND, PEBBLE-ASH, AND CINDER-ASH CONCRETE SLABS WITH 5- AND 4-BAG CEMENT CONTENTS

Left half exposed ½-submerged, right half completely submerged in sulphur water for 4¾ years. Top row: 5PS2, 5PA2, 5CA2 (half submerged), 5PS1, 5PA1, 5CA1 (completely submerged).

Cylinders are fly-ash admix specimens submerged in sulphur water for 10¾ years.

FIG. 10 (BOTTOM LEFT)—TOP FOUR SPECIMENS (29S, 29S, 29P, 29P) HALF-SUBMERGED IN FRESH WATER; ALL OTHER SPECIMENS HALF-SUBMERGED IN SULPHUR WATER—PERIODS OF EXPOSURE NOT UNIFORM—SEE TEXT

Center row: 35S, 35P, 35P, 29S, 29S, 29P, 29P. Bottom row at right: 6PPA1, 6PPA3.

FIG. 11—CAISSON CONCRETE SPECIMENS SHOWING EXTENT OF DISINTEGRATION AFTER 10½ YEARS EXPOSURE IN DIFFERENT CURING MEDIA

TABLE 3—MIX DATA

Date Made	Mkd.	Nominal Mix	Cement Content Bags per Cu. Yd.	Water Content Gal.	Slump In.
12-13-35	5PS	1 Cement—2.27 Sand—3.70 Pebbles	5	7.5	¼
12-13-35	4PS	1 Cement—3.04 Sand—4.57 Pebbles	4	9.5	¾
1- 7-36	5PA	1 Cement—2.50 Ash —3.60 Pebbles	5	13.1	7¼
1- 7-36	4PA	1 Cement—2.50 Ash —5.64 Pebbles	4	13.4	6½
10-13-35	5CA	1 Cement—2.0 Ash —4.59 Cinders	5	12.9	3½
10-13-35	4CA	1 Cement—2.5 Ash —5.81 Cinders	4	16.7	3¾

the more severe exposure of half in sulphur water and half in air, for different periods, as follows:

29S and 29P submerged 4½ years—half-submerged 8 years in fresh water
" " " " " " — " " 8 " " sulphur water
35S " 35P " 8 " — " " 4½ " " " "
6PPA " 0 " — " " 4½ " " " "

The other four specimens have been exposed half-submerged for five years, with the molds left on. One steel mold and one wood mold have been removed for the purpose of this picture. Considering the fact that the wood mold swelled, to the extent that it had no contact with a good part of the concrete, it still provided remarkable protection. Data on this mix, as well as the 6PPA specimens, are given in Table 2.

Fig. 11 shows that the amount of disintegration which has occurred at the sides and bottom of the specimens, after 10½ years of total submersion in the sulphur water, is slight. The worst disintegration is at the top edges, as shown by the other illustrations.

CONCLUSIONS

The rich mixes and low water ratios have a better rating than the leaner mixes and higher water ratios, but these factors alone do not seem to give sufficient protection.

The best ratings go to the special cements, Lumnite, Trass, and Super; or mixes with some form of volcanic ash substituted for part of the cement or sand, or used simply as an admixture; e.g., fly-ash admix, Pumicite substitution.

In the case of Lumnite cement, the heat generated during the hydration period, would have to be taken into consideration, especially in caissons or other massive concrete foundations.

During the progress of this Delray job, thermocouples were placed in two of the caissons, and it was found that the temperature developed in the concrete jumped from 68° F. at time of placing, to 137° F.

38 hours later, and it was 16 days before the temperature receded to 78° F.

There seems to be definite evidence that a good tight caisson lining will afford good protection to the concrete against the attack of sulphur water.

RECOMMENDATIONS

If we had foundations to build today under similar conditions, we would, as a result of these tests, proceed as follows:

1. Seal off the sulphur water by grouting the rock seams if possible, using a volcanic ash or fly-ash in the grout.

2. Use a good, tight caisson lining of steel or wood, preferably the latter, made from 3-in. tongue and groove lumber of a first grade, with a protective coating of linseed oil.

3. Specify a modified, low-heat cement or standard portland cement with up to 35 per cent volcanic ash, or fly-ash, added to the mix.

4. Reduce the sand ratio to a minimum, making up the deficiency in fines with more volcanic ash or fly-ash.

5. Specify a minimum cement content of 6½ bags per cu. yd. and a water-cement ratio that will produce a concrete of the least slump consistent with good placing.

Acknowledgment is due Claire H. Fellows, in charge of the Chemical Division of The Detroit Edison Co. Research Laboratory, for his valued assistance in planning the tests; also various other members of the research laboratory for the great amount of work they have done, and are continuing to do in compiling the data on this extended program.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

Unusual Concrete Roof of Hollow Girders and Precast Slabs*

By Homer M. Hadley†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The concrete roof described was designed to be wholly precast and of minimum weight. The contractor preferred to pour the thin-sectioned hollow girders in place. His simple and effective methods of placement of the girders' 1½-inch top and bottom slabs and for the 2-inch side and cross-webs produced thoroughly satisfactory results.

The new sack house of Superior Portland Cement, Inc. at its mill at Concrete, Washington is a long narrow building, 31 ft.-6 in. by 192 ft. in overall dimensions occupying a strip of ground between railroad tracks leading to the adjoining packing house. For 80 ft. at this packing house end the sack house is two stories high, the remainder one story. Loading platforms at first floor level, 4 ft. 3 in. wide, extend the full length of the building on both sides at customary elevation above the rails. Except in two of the side panels which are all 16 ft. long and at the narrow end of the building, there are sliding doors between columns throughout the first story.

Fig. 1 shows typical sections of the building as originally designed with light steel framing and with corrugated sheets for wall and roof. Note that in the one-story portion of the building the steel trusses and roof cantilever over the loading platforms. Because of possible accumulation of dust the roofs were designed for a live load of 35 lbs. per sq. ft.

Earlier work with experimental thin-webbed precast hollow beams which had shown excellent—in fact what might be regarded as extra-

*Received by the Institute Oct. 28, 1940.
†Regional Structural Engineer, Portland Cement Association, Seattle, Washington.

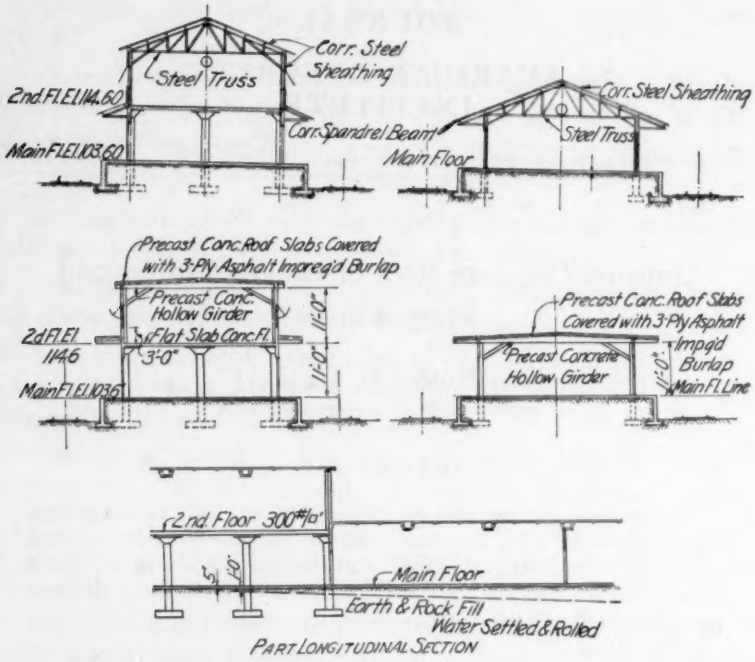

FIG. 1 (TOP)—TYPICAL SECTIONS OF SACK HOUSE AS DESIGNED FOR STEEL TRUSSES AND CORRUGATED SHEATHING

FIG. 2—TYPICAL SECTIONS OF ALTERNATE, CONCRETE, DESIGN

ordinary—load-carrying capacities led the writer to propose that a second design be prepared and alternate bids be invited on a design using concrete instead of steel, substituting 6-inch wall sections for the corrugated siding, precast hollow girders for the steel trusses, and large precast ribbed slabs, 4 ft. by 14 ft. in the typical case, for the intermediate framing. Fig. 2 shows typical sections of this design. Again note that in the one-story portion of the building the girders and roof cantilever over the loading platforms. For all concrete work at the site Superior Portland Cement, Inc. offered to furnish the necessary cement and aggregates to the contractor who was to provide form lumber, reinforcement, and other materials, and was to perform all work. To this extent the concrete design was encouraged and subsidized.

For various reasons most contractors regarded the precast design unfavorably. The interference of the girder with the spandrel beams in the one-story portion of the building is definitely objectionable,

although it could have been variously overcome. These girders although 41 ft. long weighed less than 4½ tons and to swing them to miss the spandrel beams would not have been too grievous an undertaking. However, suspicion of the unknown and unfamiliar is frequently justified and assuredly this design could only be regarded as of that category.

Bids in general reflected the prevailing skepticism but in the case of the lowest bids on the two designs less than $100 separated them and there seemed no question as to the desirability of awarding the contract for the concrete design. The contractor was most desirous however that he be permitted to cast the girders in place. Since there was no valid reason why this should not be done if he wanted to do so the change was approved and the work was executed accordingly. A decidedly curious appearance attaches to the girder section with its thin webs and flanges and largely needless knee-braces when considered as a monolithic unit, since pouring in situ practically eliminates the question of dead weight on short spans. However the simplicity of the methods by which what undoubtedly appears to be a difficult construction was built is of interest. So likewise is a statement on the appearance of the girders after the forms were stripped and on their subsequent performance.

Fig. 3 shows the details of the girder, columns and knee-braces as designed for precast construction with the object of holding weight to a minimum. The 2-ft. breadth of the girder materially shortens the span of the intervening precast roof slabs and at the same time causes the girder to receive directly and to carry without lateral transference one-eighth of the total panel live load. It has the further advantage of providing lateral stiffness, which in a precast member is a much-to-be-desired property. The only construction departure from this design was that the girder sections at the column heads were made solid. The 2-in. thickness of girder webs and diaphragms, the 1½-in. thickness of top and bottom slabs, the increased bottom slab thickness for the tension steel were all built as shown in the section. Only one inch of cover was provided over the main reinforcing bars. This is less than is ordinarily given but on the other hand it is precisely 1 inch more than the alternate and competing steel trusses had on their bottom flanges so no grievous compunctions were felt on thus departing from standard specifications. The main reinforcement is likewise unconventionally spaced and crowded but has ample end anchorage and is held by numerous stirrups. Some tests presently to be published indicate the structural adequacy if

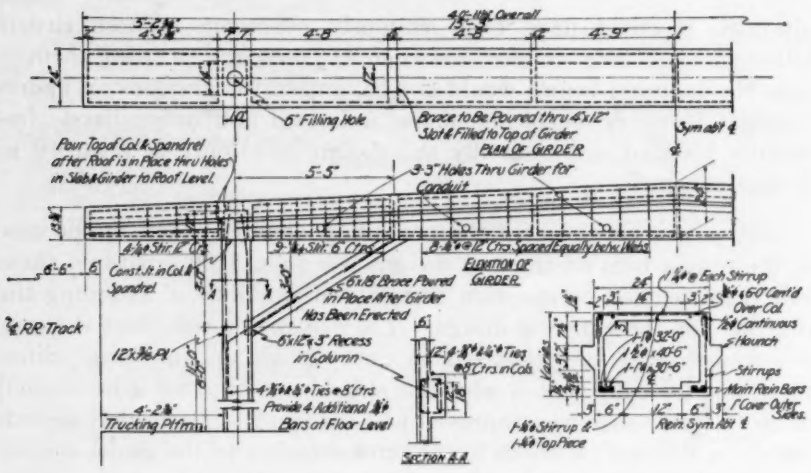
6" Filling Hole
Brace to Be Poured thru 4"x12" Slot & Filled to Top of Girder
PLAN OF GIRDER
Pour Top of Col. & Spandrel after Roof is in Place thru Holes in Slab & Girder to Roof Level
3-3" Holes Thru Girder for Conduit
ELEVATION OF GIRDER
6"x18" Brace Poured in Place After Girder Has Been Erected
6"x12"x3" Recess in Column
Const. It in Col. & Spandrel
R.R. Track
Trucking Platform
Provide 4 Additional Bars at Floor Level
Section A-A
Stirrups
Main Rein. Bars
Rein. Sym. about ℄

FIG. 3—DETAILS OF GIRDER, COLUMNS AND KNEE BRACES

not orthodoxy of this arrangement. Note that the tops of the web stirrups are hooked in the plane of the webs and do not interfere with the placement of the cores; also, that the top slab bars turn down 6 in. into the webs.

CONSTRUCTION

Girder construction proceeded as follows. After the outer form was built the main tension reinforcement—all straight bars—and stirrups were placed and also the forms for shaping the offsets in the bottom slab. These latter were merely rectangular frames made of 1 x 1½-in. strips blocked up 1½-in. above the bottom form. The bottom of the frame made a screed to which the 1½-in. slab thickness was struck off while the top of the frame determined the 3-in. thickness wanted at the sides of the bottom slab where the main reinforcement is located. The bottom slab concrete was then placed to the required 1½-in. and 3-in. thickness. Inasmuch as this operation was no more than spreading a small amount of concrete over the open bottom of a beam form 2 ft. wide, it is apparent that no difficulty arises here.

The next step was to place the main core forms. These had previously been made up in a nearby work yard. There were eleven girders in all, symmetrical about their midspan points, identical in the portions between their supporting columns and varying only in the end conditions of those used in the one-story and two-story portions of the building. Consequently of the 66 core forms required for the

main central part of girders there were only three different sizes, 22 of each. All cores were made of ¼-in. plywood with good but not painstaking carpentry. They were to be buried up and did not require painstaking carpentry; neither did they require the finest quality of material. They were open on the bottom where they set down upon the marginal frames and bottom slab and were simply shells of plywood braced by interior frames of light wood. One of the accompanying views clearly shows the interior construction of one of the cores.

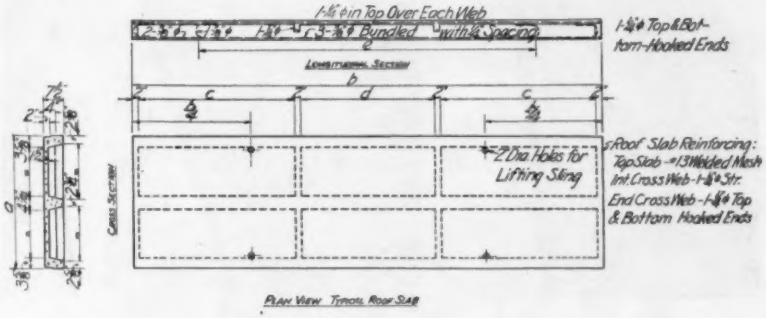
1-½" in Top Over Each Web
1-¼" Top & Bottom-Hooked Ends
Longitudinal Section
2" Dia. Holes for Lifting Sling
Roof Slab Reinforcing:
Top Slab - 1 Welded Mesh
Int. Cross Web - 1-¼" Str.
End Cross Web - 1-¼" Top & Bottom Hooked Ends
Cross Section
Plan View Typical Roof Slab

FIG. 4—DETAILS OF TYPICAL PRECAST ROOF SLABS

When these cores were set the reinforcement of the top slab and the longitudinal top bars at the columns were placed. Then concreting was resumed, the concrete being shoveled or dumped directly from the buggies on top of the cores, whence it was worked laterally into the webs and diaphragms. To fill webs only 2 in. thick would normally be regarded as difficult construction. However thin webs in combination with open top fillets are much more easily filled. Here the 3-in. 45-degree fillets increased the total top breadth to 5-in., served to receive a considerable volume of concrete and directed its downward course. The webs and diaphragms were tamped with thin tamping sticks and a small amount of vibration was applied to the girder as a whole with a heavy external vibrator. After the webs had been filled the top slab was completed—which also was not difficult to do.

On removing the forms all surfaces were found to be full and complete—there were no honeycombed areas or any spots to suggest imperfect placement. The concrete used was a rich plastic mix: 1.75 bbls. high early strength cement per cu. yd., ½-in. maximum size river gravel, a natural sand and water sufficient to give a sluggish, rather

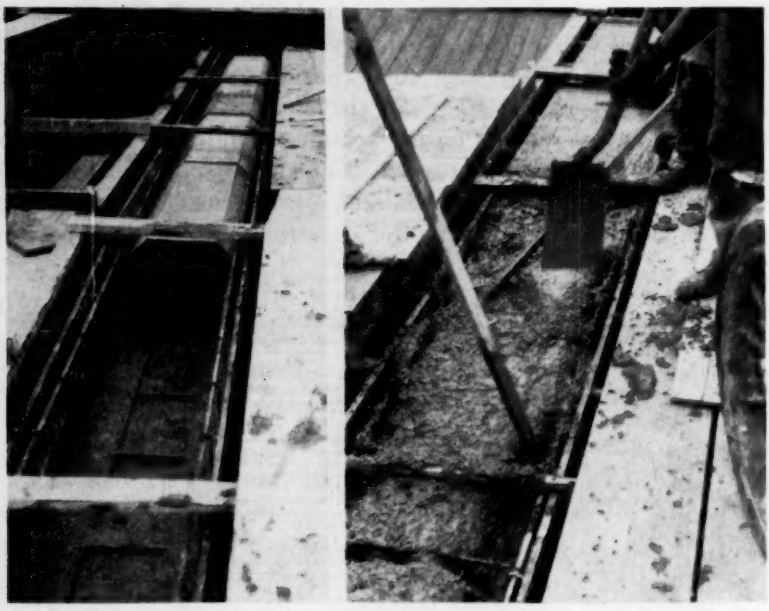

FIG. 5—HOLLOW GIRDER FORMS AND CORE—BOTTOM CONCRETE PLACED; RIGHT, PLACING CONCRETE IN RIBS AND TOP SLAB AFTER CORES AND TOP REINFORCEMENT HAD BEEN PLACED

stiff consistency. There was no reason why perfect placement should not have been obtained.

The inner forms were of course left embedded in the concrete. With plywood there is little or no expansion and with ordinary lumber that has been thoroughly wet before concrete is placed around it, there also is little or no expansion because it is in its state of maximum extension when buried. It can shrink but it cannot further expand. Therefore when it is desirable to leave forms embedded in concrete it can safely be done by having them thoroughly wet. The writer knows of half a dozen box girder bridges that demonstrate the correctness of this statement.

Details of a typical precast roof slab which spanned the 14-ft. opening between girders are shown in Fig. 4. These 4 x 14 ft. slabs are larger than are generally employed but with the panel dimensions determined by the original design of structural steel there appeared to be no simpler way of framing between the girders than directly, with ribbed slabs of this size. They were made by the Olympian

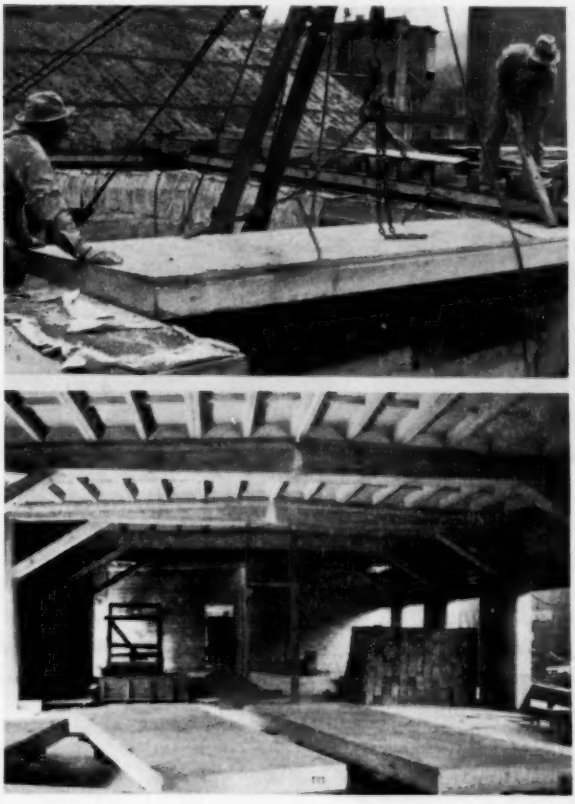

FIG. 6—SETTING A ROOF SLAB

FIG. 7—COMPLETED ROOF

Stone Co. of Seattle on a vibrating table and were truck hauled 100 miles to Concrete. It was originally intended to use light weight aggregate but it finally resulted that natural aggregates were employed. A typical slab weighed about 2,100 lbs. They were unloaded from the truck directly to the floor of the sack house by means of an overhead trolley running on an I beam. Lowered onto dollies they were rolled to the bay where they were to be used and were there set down until finally placed. Those for the two-story part were lifted to second floor and similarly distributed. The final lifting from floor to roof was accomplished very simply with a light hoisting frame near the bottom of which was a small motor driven winch and at the top a set of sheaves. Guy lines steadied and held the top in desired

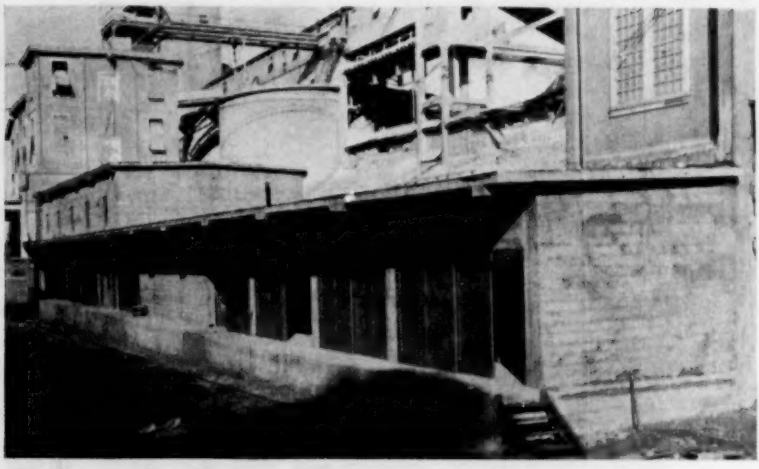

FIG. 8—THE COMPLETED SACK HOUSE

position. With this rig the slabs were lifted and in general were set down directly in their final positions on the girder corbels. After the slabs were set all joints were grouted and the upper part of the spandrel beams poured through the filling holes left in the slabs which came above the beams.

A test load of 35 lb. per sq. ft., the designed live load, was applied over a 30-ft. width of roof to subject one girder to its full working load. The loading material was cement in sacks. In distributing this with a small-wheeled hand truck, the roof slabs were subjected to concentrated wheel loads of 330 lbs. Under full load the girder deflected $\frac{5}{64}$ in. at midspan but without any hair cracking that could be observed. On removal of the load the deflection disappeared.

Western Construction Co. of Seattle was the contractor. The building was designed and was constructed under the supervision of N. H. Peterson, Chief Engineer, Superior Portland Cement, Inc. The writer was responsible for the design of the roof.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by Homer M. Hadley:

Unusual Concrete Roof of Hollow Girders and Precast Slabs*

BY N. H. PETERSON†

When it was finally decided that a new sack house was to be built at our plant, there was a prevailing opinion that we would necessarily need a steel trussed super structure and an outside steel covering for a low cost fire resistant building. Mr. Hadley's roof design looked attractive but expensive to us. The subsequent bids dispelled those fears and for obvious reasons, when the price was found to be in line, a concrete roof was favored over the steel and the unusual roof resulted.

There seemed to be a skepticism on the part of the contractor about precasting the hollow girders. A good deal of this was due to the rather limited accessibility. In the first place, the building was long and narrow and could only be approached from one end. In addition, the railroad spurs on each side of the building were almost constantly full of box cars waiting to be loaded with cement. The contractor believed handling the long 4½ ton box girders under such conditions would have been too difficult. It was therefore agreed that they be poured in place. Had we made this agreement earlier, the girders probably would have been made solid and conventional in design. The box design, however, proved itself at least equal in all respects to the solid type, not only in strength but economy.

The thin walls of the girders had a surprisingly good effect on the men tamping the concrete. They conceived the idea that it required more tamping here than is usual. Tamping is often underestimated and is seldom overdone. In our case the good tamping made itself

*A. C. I. JOURNAL, Feb. 1941; *Proceedings* V. 37, p. 453.
†Chief Engineer, Superior Portland Cement, Inc., Concrete, Wash.

quite apparent as soon as the first forms were removed. All exterior surfaces showed the fine appearance of the concrete.

No difficulty whatever was encountered in placing the precast roof slabs. They were easily and quickly handled by a crew of five men. Excellent workmanship on the part of The Olympian Stone Co. which manufactured them, enabled all to be placed without field fitting or trimming. The A frame hoisted all slabs in each bay into place except the two in the middle. These were laid on top of those adjacent and were later hoisted into place by means of a chain block attached to a wood beam set between two seven foot scaffold supports.

Incidentally one of the roof slabs that was being laid on top of the roof for center placement was accidentally dropped from a height of about 18 in. to the roof. It was gratifying to feel the solidity of the whole roof structure under the impact of the blow. The incident only substantiated the remarkable strength that was only indicated by the successful load test referred to by Mr. Hadley.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

The Ida B. Wells Low-Cost Housing Project in Chicago*

By Carl A. Metz†

SYNOPSIS

This paper records the principal elements of design and construction of a low-cost housing group of 125 buildings of fireproof construction in Chicago—1662 dwelling units, 6901 rooms—at a cost per room of $990. Reinforced concrete frame construction was used rather than masonry bearing walls and the reasons are given. Floors are clay tile filler and concrete joists with clay tile soffits and 2-in. concrete top slab.

The Ida B. Wells Housing Project, to provide 1662 dwelling units in 125 non-uniform buildings for negro occupancy on a 56-acre tract in Chicago, is chiefly interesting in the use of reinforced concrete frame rather than masonry bearing wall construction and in the choice of clay tile filler and concrete joist floors.

It was a U. S. Housing Administration job originally intended for brick masonry bearing wall and solid flat slab construction.

Three firms of architects were selected: Shaw Naess & Murphy, Theilbar & Fugard, and Nimmons Carr & Wright to design the buildings in this project. These architects employed Magnus Gundersen and myself as Metz & Gundersen to prepare the structural designs.

We had the following as a basis for our preliminary studies.

1. The foundation bearing soil was lake sand.

2. There were to be 125 buildings not all the same size and design, comprising approximately 1,600,000 sq. ft. of supported floor and roof area.

3. Buildings were to vary from two to four stories.

4. We had the designs of three other Chicago housing projects for reference if needed.

*Received by the Institute Oct. 14, 1940.
†Consulting Structural Engineer, Chicago.

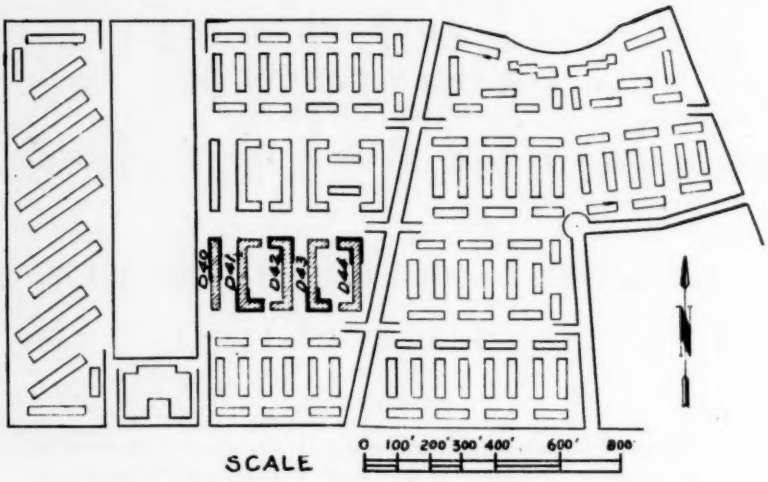

FIG. 1—KEY PLAN IDA B. WELLS HOUSING, CHICAGO

5. We had the knowledge that much trouble was being encountered with parapet walls.

6. The buildings had to be fireproof in construction.

7. The ceilings of the buildings had to be plastered (a local labor union requirement).

The sand gave us no problems regarding foundations. We used a bearing value of 4000 lb. per sq. ft.

Since the ceilings had to be plastered, we wanted a better plaster base than the solid slab could be expected to provide—the architects had told us they planned using a solid concrete slab. Thus our first problem was the selection of the type of floor construction.

Knowing Chicago as a brick and tile territory, we assumed that clay tile filler and concrete joists should be the most economical and best suited to the job.

After talking with a few large contractors, it seemed it was wrong to consider this job as one large project—each building was a separate job.

A project with 125 non-uniform buildings necessarily involved more interruptions in the steady flow of construction work than would a single building of the same total capacity. Our job seemed to be to design structures that were, first of all, sound, and, second, that would permit the greatest possible standardization combined with the fewest interruptions in the construction procedure. We therefore prepared typical designs using: (1) A solid flat slab design plastered; (2) A removable metal pan form and concrete joists with

metal lath and plaster; (3) A 16 in. wide by 4 in. thick clay tile filler on 20 in. centers with tile soffits under the joists, and plastered; (4) An 18 in. wide by 4 in. thick clay tile filler on 22 in. centers with tile soffits under the joists and plastered; (5) A 20 in. wide by 4 in. thick clay tile filler on 24 in. centers with tile soffits under the joists and plastered.

These five designs were submitted to three contractors, who had had experience building low-cost housing units, for costs. Two of the three contractors found the 16 in. wide by 4 in. thick tile filler with 2 in. top slab to be the cheapest (Table 1). We disposed of the floor problem in deciding on this type. Here are the tabulated cost estimates as submitted by the contractors:

TABLE 1—CONTRACTORS COST IN CENTS PER SQ. FT. FLOOR AREA

	Contractor No. 1	Contractor No. 2	Contractor No. 3
Scheme 1—Solid slab	54.2	61.4	60.5
Scheme 2—Metal pan and joists	56.5	58.5	67.4
Scheme 3—16"x4" tile and joists	51.0	55.0	65.2
Scheme 4—18"x4" tile and joists	52.8	55.5	64.6
Scheme 5—20"x4" tile and joists	52.8	55.3	63.8

This brought us to the problem of slab supports at the exterior walls. Most previous jobs had been wall bearing and the architects and U. S. H. A. engineers had indicated to us that this job was to be wall bearing. We had in mind a complete concrete frame. We wanted to get away from the trouble experienced on wall bearing jobs with cracked coping walls. Due to expansion, poorly built walls, curling of the concrete roof slabs at the corners of the buildings, or possibly to a combination of all these causes, parapet walls were pushed out and up at the corners of the buildings at the roof line. About half the buildings in the Jane Addams group (1027 dwelling units) in Chicago now have the parapet walls so damaged.

About two years previous to this work, I had designed a 3-story school building using the tile and joist floor slabs and wall bearing construction. The job was let to a contractor, who, after he had started work in the fall of the year, came to me and asked permission to make the job skeleton frame. He asked for no additional money and his reason for the request was that he wanted to have all his concrete placed before freezing weather set in. I naturally granted the request and the job was so built. After the job was finished he told me without showing me any figures that the change saved him time which meant money to him.

With a wall bearing job, walls would have to be 12½ in. and 16½ in. thick. Masons would have to build a scaffold from the ground, build a wall to the next floor level, remove their scaffold, and stop until the next floor was framed and poured. They would then move their scaffold to the next level and build the wall to the level of another floor, again remove their scaffold, wait another interval, and so on to the roof.

Consider what this starting and stopping means. The flow of materials has to be stopped and every mason must gather up his tools and go to another building. There is extra labor in handling scaffolding and in the wrecking of form work, which must all be removed through windows, if possible, or down stair openings. With a reinforced concrete frame, the wrecking of forms is easier as it is open on four sides and lumber can be moved up or dropped at any point around the building.

We believed that the framed structure was as desirable in the two-story building as in four-story building. Frames would all be built complete to the roof. Then the brick mason would begin his brick work by hanging a scaffold from the roof and working his way up to the top, the steel sash being installed as the work progressed. With the frame, we would cut the thickness of walls from 12½ in. to 8½ in. with waterproofing, metal furring, and lath and plaster on the inside of the walls for a total thickness of 10 in.

To prevent the movement of parapet walls at the corners we reasoned that a corner concrete column with spandrel beams would surely prevent curling of the roof slab. We hoped that this frame would prevent all trouble previously encountered with these parapet walls. (I do not like to speak too soon, but to date no damage has appeared at these points and some of the buildings have gone through one summer and two winter seasons.)

We recommended to the architects that the framed structures be used and the exterior walls be cut from 12½ in. to 8½ in. We were certain that the time saved in construction would more than offset any estimated extra cost of construction. Our recommendations were accepted.

The advantages which we feel we have in the framed structure over the wall bearing job are as follows:

1. No additional first cost. (Final contract figures seem to bear this out.)

2. Less maintenance. (If we have no trouble with parapet walls.)

3. Admittedly a better job structurally.

4. No interruptions in building the complete frame.

5. No interruption to the brick layer on the enclosing walls.

6. Easier and cheaper wrecking of forms.

7. Speedier construction. (The general contractor has reassured me on this point.)

8. Less brick masonry.

9. No splashing of concrete on the outside surface of the brick walls. In fact there was just enough mortar splashing on the brickwork to appeal to the architects so that they omitted the cleaning down of the brick walls.

Type of floor construction and the choice between frame or wall bearing construction were the two major problems on this project. This settled, all we had to do was to design the joists, beams, columns, walls, and footings, which procedure is nothing new to concrete designers.

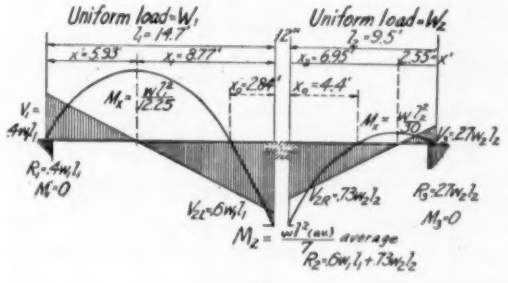

FIG. 1

For the slab design we worked out a typical joist assuming restraint at the center column. This moment or shear diagram is shown (Fig. 1). This gave a maximum positive moment of $\frac{wl^2}{12.25}$ for the long span and $\frac{wl^2}{30}$ for the short span and a negative moment over the center support of $\frac{wl^2}{7}$ (average). The maximum shear was $.6wl$ at the center support. This shear was taken care of by starting the first tile filler, which was a 12 in. wide tile instead of the regular 16 in. tile, at a distance from this support which developed the shear value.

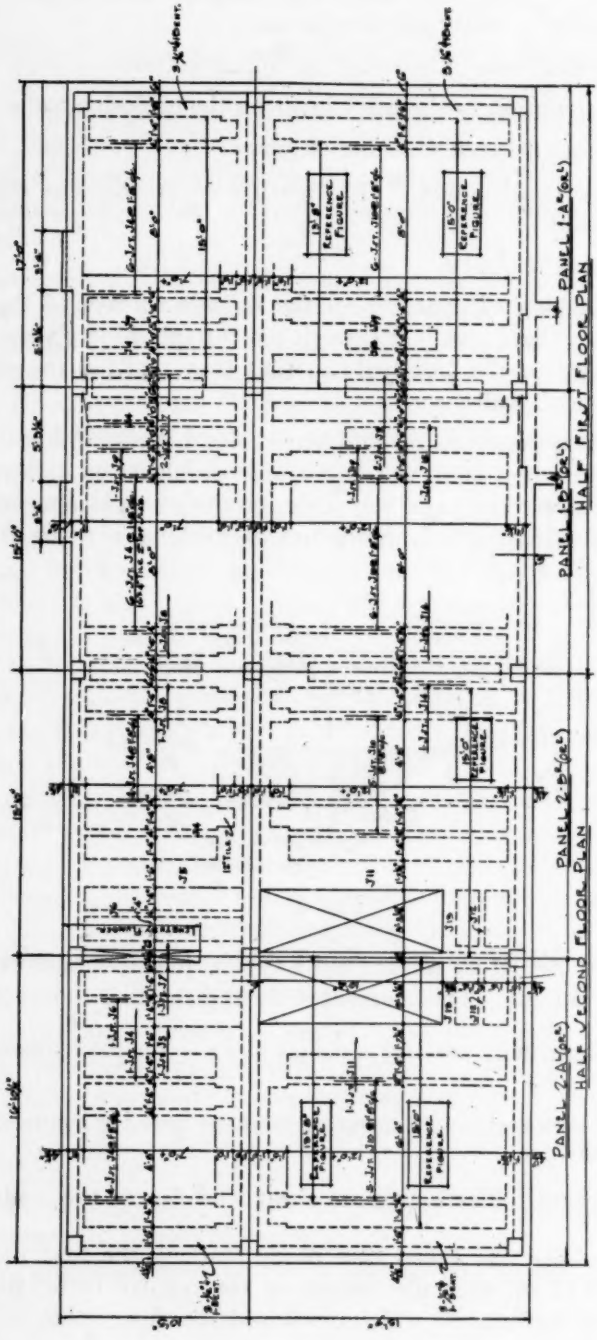

FIG. 3—TYPICAL FLOOR JOIST FRAMING

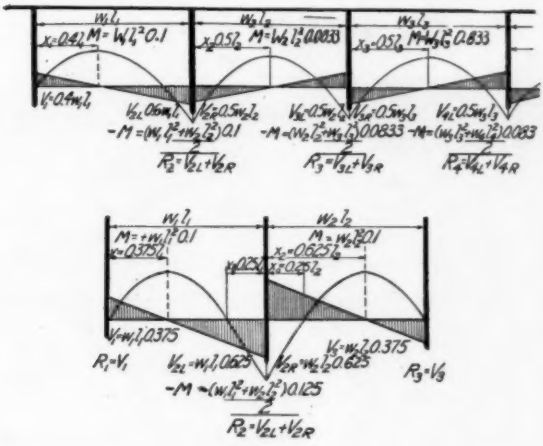

FIG. 4

For the typical slab the loads were:

Finish	2 lb.
Slab	47 lb.
Plaster	5 lb.
Live load	40 lb.
Total	94 lb.

Joist spacing was 1 ft.-8 in.; load per joist was $94 \times 1.67 = 157$ lb. per lin. ft.

With the 6 in. slab and a $\frac{5}{8}$ in. tile soffit the d was $4\frac{1}{2}$ in.

$$M = \frac{157 \times 15.2^2}{12.25} = 2960 \text{ ft. lb.} \quad A_s = \frac{2960 \times 12}{.9 \times 4.5 \times 20000} = .44 \text{ sq. in.}$$

We used 2-$\frac{1}{2}$ sq. bars giving an area of .50 sq. in. Moment coefficient was $\frac{157 \times 15.2^2}{.50 \times 1500 \times 4.5} = 10.7$ or $M = \frac{wl^2}{10.7}$

$$\text{Neg. } M = \frac{157 \times \left(\frac{15.2 + 10}{2}\right)^2}{7} = 3560 \text{ ft. lb.} \quad A_s = \frac{3560}{1500 \times 4.5} =$$

.525 sq. in.

We used two $\frac{1}{2}$-in. sq. bars one bent up from each span.

For the short span joists we used one $\frac{1}{2}$-in. sq. bent over support and one $\frac{3}{8}$ in. rd. bar straight. This gave us a moment coefficient of $\frac{157 \times 10^2}{0.36 \times 1500 \times 4.5} = 6.5$ or $M = \frac{wl^2}{6.5}$ while $\frac{wl^2}{30}$ was required.

However, from a practical standpoint we could not put in any less steel. The bent bars were extended 4 ft. either side the center sup-

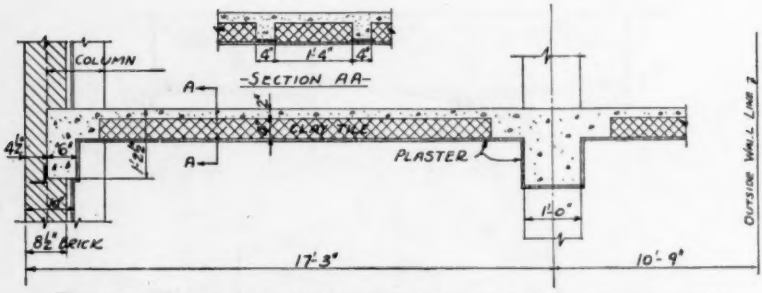

FIG. 5—TYPICAL SECTION THROUGH FLOOR CONSTRUCTION

port. In the short span this did not quite reach the figured point of moment change, but we assumed that the temperature steel in the top slab and tension in the concrete would take this small tension value at this point. All unit values as given in the A. C. I. Building Code for 2500 lb. concrete were used.

The beams were figured according to the diagrams, Fig. 4, all as recommended in the A. C. I. Code.

The new Chicago Ordinance follows the A. C. I. Code for concrete design except for columns. Chicago allows a minimum column of 120 sq. in. and a minimum of four ¾ in. round bars tied with ¼ in. round bars—12 in. on center.

The Chicago formula for tied columns is

$$P = A\ (0.2\,f'_c + 0.9\,f_s p). \qquad A = \text{gross area.}$$

Using a typical 12 in. square column with four ¾° vertical rods.

$$P = 144\ (0.2 \times 2500 + 0.9 \times 15000 \times .01225) = 95{,}750 \text{ lb.}$$

$$p = \frac{4 \times 0.4418}{144} = .01225$$

The footings were all designed for 4000 lb. soil bearing value and 2500 lb. concrete.

In general we gave special consideration to standardization of design. The surest way to reduce costs is to make all elements of the structural design as simple and typical as possible. This idea, of course, is not new by any means, but I am inclined to believe that its importance is not sufficiently realized by the average practicing architect and engineer.

On a large job such as this the general contractor has a form building shop with modern equipment. If a column form, for instance, is stripped from one building and is exactly the size of a column in an-

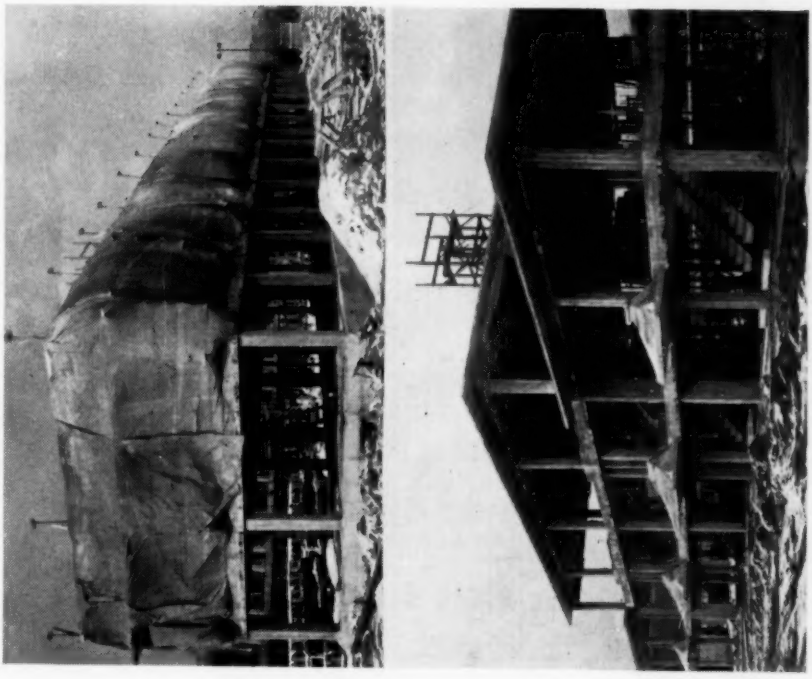

FIG. 6, AND 7, 8—CONSTRUCTION VIEWS
IDA B. WELLS HOUSING BUILDINGS

TABLE 2—COMPARATIVE DATA ON NINE HOUSING PROJECTS

Name and Location of Housing Development	Total Cost Construction Dwelling and Non-Dwelling	Total No. of Units	Const. Cost Per Unit	Total Number of Rooms	Const. Cost Per Room	Type of Finance	Type of Construction
Ida B. Wells, Chicago	$ 6,841,926	1,662	$4,117	6,901	$ 990	USHA	Conc. Frame, Clay Tile & Conc. Fls.
Julia Lathrop, Chicago	4,499,863	925	4,860	3,254	1,380	USHA	Wall Bearing Conc. Fls.
Trumbull Park, Chicago	2,512,812	462	5,440	1,733	1,450	USHA	Wall Bearing Conc. Fls.
Jane Addams, Chicago	4,942,208	1,027	4,810	3,571	1,384	USHA	Wall Bearing Conc. Fls.
Red Hook, New York City	10,304,909	2,583	4,000	10,657	963	USHA	Wall Bearing Conc. Fls.
Williamsburg, Brooklyn	8,490,885	1,662	5,222	5,688	1,490	USHA	Concrete Skeleton and Fls.
Old Harbor, Boston	5,257,919	1,016	5,180	3,794	1,390	USHA	Wall Bearing Conc. Fls.
Chatham, Chicago	2,832,543	554	5,120	2,207	1,280	FHA	Wall Brg. Conc. 1st Fl. Wd. Const. Bal.
	Total Cost Dwellings Only					Cost Per Cu. Ft.	
Ida B. Wells, Chicago	5,723,000	1,662	3,450	6,901	830	0.364	

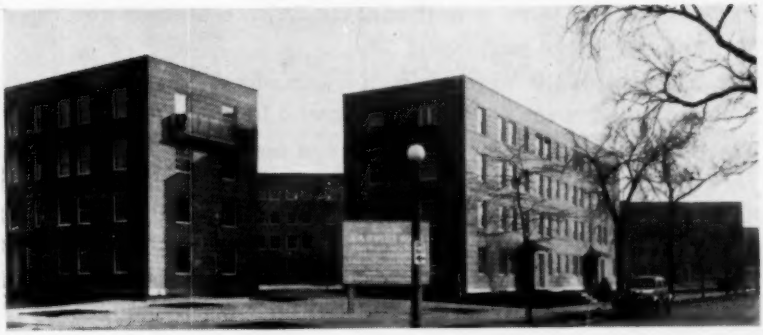

FIG. 9—COMPLETED UNITS, IDA B. WELLS HOUSING PROJECT

other building, but has a slightly different beam cut, that form must be re-worked either in the field or in this shop before it can be re-used.

We made all slabs the same as far as possible, columns were of two sizes only, 10x12 and 12x12, the spandrel beams were all the same, 6 in. by 14½ in., and the interior beams were all the same size as nearly as possible. The variable was the reinforcing steel. Typically the slab joists were 4 in. wide with two bars, the spandrel beams 6 in. wide with two bars, the interior beams 12 in. wide with four bars, and the columns with four vertical bars and ¼ in. round ties, 12 in. on centers.

I may say at this point that the architects cooperated 100 per cent with us in our investigations and decisions. There was opposition from the U. S. H. A. engineers who were almost insistent upon wall bearing construction, at least for the two-story structures. We held our ground and the buildings are all framed structures.

I can supply no actual figures proving that the concrete frame is more economical than the wall bearing job, but I do know that the bids were about $500,000 under the original estimate and that this job ran about $400 per room cheaper than other U. S. H. A. Chicago jobs. In all my private design work, I very seldom use a masonry wall to support concrete floors. Where at all possible I design a complete concrete frame.

Summarizing the job: we designed for a 2500 lb. concrete and 20,000 lb. reinforcing steel. The contractor asked to use a slag coarse aggregate which we allowed, and his cylinders all tested between 3000 and 4500 lb. at 28 days.

Slabs had 16 in. tile filler with 4 in. wide joists, reinforced with two bars, one bent, with 2 in. of concrete over the tile giving a total

thickness of 6 in.; ¼ in. round bars on 12 in. centers in the top slab at right angles to the joist steel.

The basement walls were 10 in. thick reinforced to carry the spandrel column loads over a continuous spread footing.

The project has 1,600,000 square feet of floor area; used 10,000 tons of clay tile fillers and soffits; 46,000 cu. yds. of concrete in the frames; 3,000 tons of reinforcing steel. There are 124 separate buildings and the administration and boiler-house; 1,662 living units; 6,901 rooms.

Steel sash, 8½ common brick walls, 2 in. solid plaster partitions, steel stairs, asphalt tile floors, metal lath and plaster on outside wall and plaster ceilings.

The smallest building in the group is 26 ft. 2 in. wide by 65 ft. 5 in. long, 2 stories high. The largest is 27 ft. by 270 ft. 8 in. long and four stories high. Story heights are 8 ft. 9 in.

The contractor had a central concrete mixing plant from which he hauled the concrete in mixing trucks to the various buildings. We had at all times a very uniform grade of concrete at all points of deposit.

We started actual working drawings on April 1, and finished June 15, 1939. This design included a central power plant and administration building. Work was started by Thompson-Starrett Co. Aug. 24, 1939. They were stopped by a strike on Aug. 25, 1939. Work was resumed on October 24, 1939 and the roof of the last building was poured on July 3, 1940.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by Carl A. Metz:

The Ida B. Wells Low-Cost Housing Project in Chicago*

BY A. M. KORSMO AND THE AUTHOR

BY MR. KORSMO†

This interesting paper by Mr. Metz treats of two important structural factors in USHA housing design; namely, the floor system and the ever-recurring question of skeleton construction versus wall-bearing construction. As indicated by the floor plan shown in his paper, the typical buildings are regular in arrangement, and, as he said, once the framing system has been determined, the structural design computations are simple.

The preparation of preliminary alternate designs for several types of floor slabs and obtaining comparative cost estimates from three representative local contractors, as Mr. Metz illustrated in Table 1, is to be highly commended. That procedure has been advocated by USHA throughout its program. The relative costs which he has shown are practically the same as those which we have obtained in many other cities. Only when plastered ceilings are required under solid concrete slabs, which is the case in Chicago, have tile and concrete joist floors proved to be the cheaper system.

The author's statements, "U.S.H.A. engineers had indicated to us that this job was to be wall bearing," and also, "U.S.H.A. engineers were almost insistent upon wall bearing construction" indicated that there was a misunderstanding regarding the functions of the USHA. The USHA is a loaning and supervising agency and does not dictate either the type of design or the materials of construction that shall be used. This is the responsibility of the Local Housing Authority, which

*A. C. I. JOURNAL, Feb. 1941; *Proceedings* V. 37, p. 461.
†Principal Structural Engineer, USHA, Washington, D. C.

owns and operates the project; however, we are charged, under the provisions of the Housing Act, with seeing that loans, grants, and annual contributions are made only for such low-rent housing projects as we find are undertaken in a manner that will not be of elaborate or expensive design or material, and also that economy will be promoted both in construction and administration. We, therefore, do make suggestions and recommendations, one of the most important of which is the need of making comparative estimates, before deciding on the design to be used. The experience, gained by USHA in the past seven years of building several hundred projects, should be of value to architects and engineers designing new USHA projects; particularly, when they are designing one for the first time.

Regarding the relative costs of skeleton frame and wall-bearing structures, the author stated, "I can supply no actual figures proving that the concrete frame is more economical than the wall bearing job" indicating that the same careful analysis was not made as for the selection of the floor system. The USHA, however, did make a comparative study on the Ida B. Wells Project in cooperation with one of Chicago's leading contractors who had built one housing project. His figures showed a saving of $27 per dwelling unit for the wall-bearing design of 2-story buildings, and about equal cost for the 3-story buildings. We were not aware that these figures were questioned.

The data of Table 2 cannot be used without modification for estimating the relative cost of skeleton frame and wall-bearing structures. In the first place, the Julia Lathrop, Trumbull Park, Jane Addams, Willamsburg, and Old Harbor Village Projects were not financed by USHA. They were financed by the former Housing Division of PWA, under a different program and policy. They were built in the pioneer days of slum-clearance housing, and contained many standards of design and planning which have since been relaxed or eliminated by the USHA minimum requirements for low-cost housing. This is best illustrated by stating that the average cost per dwelling unit under the USHA program is 26 per cent lower than it was under the PWA program. Secondly, the cost figures are based upon the total cost of both dwelling and non-dwelling facilities. Non-dwelling costs include excess foundations, community houses, stores, landscaping, paving, utility mains, etc., all of which distort any true comparison of the net costs of the framing. For instance, the Ida B. Wells Project had the simplest and most economical type of spread concrete foundations on sand. The Red Hook Project in New York, on the other hand, was built on a cinder fill 20 to 25 feet deep and required 35-ft. concrete pile foundations throughout. Furthermore, the room count

in the PWA program was made on a different basis than is now used by USHA.

The one project cited in Table 2 which is in the concurrent USHA program is the Red Hook Project. Note that it not only is a wall-bearing job, but that, despite its being six stories high, having elevators and expensive foundations, it was also of considerably less total cost.

It is believed that the data in Table A afford a more reliable comparison of the costs of the two types of construction:

TABLE A—USHA PROJECTS

Skeleton			Wall Bearing		
Job No.	Total Rooms	Net Constr. Cost Per Room	Job No.	Total Rooms	Net Constr. Cost Per Room
Ill-2-1	6901	$ 800	NY-5-1	10649½	$ 693
Mass-2-1	4733½	689	NY-5-2	12949½	649
Mass-2-2	3865	683	Mass-2-4	1183½	753
Mass-2-3	4311½	672	Mass-2-7	1998	686
NJ-2-1	2241	734	NJ-9-1	2032	715
NJ-2-2	991½	715	NJ-9-2	2026½	674
Totals	23043½	$4293		30839	$4170
Average		$ 716			$ 695

$\frac{(716 - 695)\ 23043\frac{1}{2}}{4280^*}$ = 113 additional dwelling units.

*Average total development cost per unit for 103 projects built in 1940.

The comparative costs, given as "net construction costs per room," include only the construction of the dwelling units and their wiring, plumbing, heating, and dwelling facilities. While a comparison on this basis is not absolutely correct for establishing differences in the framing costs, it does eliminate the large variables included in "total construction costs" such as I have already cited.

Note an average saving of $21 per room for the wall-bearing type. If this represents a saving due to type of framing, then the choice of a wall-bearing design for these six projects of skeleton design would have made 113 additional dwelling units available. On the basis of comparative estimates taken in many representative cities we have found that all 2-story buildings, and usually the 3-story buildings, have been cheaper with wall-bearing construction, depending on local building code requirements as to wall thickness, etc.

It is a well-known fact that many contractors prefer skeleton construction and we have repeatedly heard their arguments for it, which are similar to the nine which the author has set forth; but judging from their own estimates, and also the actual contract unit room costs in our table, the wall-bearing type usually is the cheaper.

In further support of the skeleton design, the author stated that "the bids were about $500,000 under the original estimate." Such a

comparison is of no value. The spread between the high and low bidders on this project was more than $671,000 and with the great number of alternates taken (32 to be exact) almost any one of the contractors bidding could have been awarded the job, depending on the alternates accepted. Architectural planning, and the plumbing and heating design affect the final capital cost more than the skeleton frame. A much more accurate comparison is the recorded cost of $3320 net construction cost per dwelling unit on the Ida B. Wells Project compared to the $2585 obtained on 103 other USHA projects, predominantly wall bearing, built during the last six months of 1940.

It is true that we had some cracking in masonry bearing walls under the PWA program, but the extent of that trouble has been grossly exaggerated. The projects which experienced such cracking were very much in the minority of the total number of projects built, and the troubles were often traced to causes other than that type of slab support. Parapet walls are a source of trouble on skeleton as well as wall-bearing designs and the causes can often be attributed to the use of highly absorbent brick, improper flashings and faulty jointing of copings.

Since the inception of the USHA program no serious cracking troubles in masonry bearing walls have been experienced, probably because of innovations which have been adopted such as continuous reinforced spandrel sections integral with the slab and special added reinforcement in the slabs at external corners, which resist curling action of slabs at the corners. On many projects the parapets have been entirely eliminated, which may be the proper solution of this problem, particularly slum-clearance housing.

In conclusion I wish to emphasize:

(1) the USHA makes no recommendation for one type of construction in preference to another, except on the basis of economy in construction and maintenance;

(2) both wall-bearing and skeleton construction are being used on our projects with such economy;

(3) the choice of the economical type should be justified by analysis of alternate designs and authentic cost estimates;

(4) USHA records, and general construction practice, have consistently indicated that wall-bearing construction is cheaper than skeleton construction for 2-story buildings and usually for 3-story buildings;

(5) either type is substantiated by previous use over long periods, both subject to some degree of cracking and parapet wall troubles which are not considered serious in either case.

It does not seem consistent that an economical type of construction

which has been used all these years and has given such generally satisfactory service should now be arbitrarily discarded. We are continually striving for lower costs and we welcome any new framing designs and materials of construction which will accomplish that end. As Administrator Straus has repeatedly stated, functional design, sound low-cost construction and efficient operation are the three factors that determine the real cost of housing—the cost of using a house—the cost of living in a house.

AUTHOR'S CLOSURE

The remarks by Mr. Korsmo are very interesting and I believe sincere. I am glad he chose to reply and I hope some information of value can be had from our discussions.

First, about the cracking of parapet walls. Mr. Korsmo states that this trouble has been grossly exaggerated. From information which I have about jobs out of Chicago and from personal knowledge of the trouble experienced on the other Chicago housing projects, I still believe it is a serious problem and one which must be given very close attention.

As to comparative costs of wall bearing over skeleton construction, I cannot be much disturbed about a comparison of the Wells job with jobs in New York, New Jersey, and Massachusetts. A truer comparison could have been reached had Mr. Korsmo chosen to compare the Wells job with the other three Chicago jobs which were built under similar conditions as to place, labor and materials.

Every type of construction has its place as determined by careful analysis. There may be differences of opinion as to the place, and I still believe we chose the right construction for the right place.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

Design of Chicago's Initial System of Subways*

By P. F. Girard†

SYNOPSIS

The Chicago subway design work was started in October 1938, and at the end of 1940 the tunnel work is 85 per cent and the station work 35 per cent complete. The entire project is essentially one of tunnel construction.

Conditions at the start of the project and the design work that has been accomplished since that time are described. First a general description of the work and brief review of some of the essential structures. The remainder of the paper describes the structural design of the work, with special emphasis on the concrete design of the different tunnel sections.

Construction of Chicago's "Initial System of Subways," a $46,000,000 project financed jointly by the City of Chicago and the Public Works Administration of the Federal government, is now in its final stages. This addition to Chicago's local transportation facilities is a 7.7 mile, double tube, deep-level subway and constitutes the first stage of a city-wide network of subways, the other units of which are to be built in three later stages. The location of this system is as follows:

ROUTES AND PRELIMINARY STUDIES

The State Street Subway is from a connection with the north side division of the elevated near the intersection of North and Clybourn Avenues; via Clybourn Avenue, Division Street and State Street to a connection with the structure of the south side division of the elevated near 14th Street. The length of this route is 4.4 miles. The estimated cost of construction is $26,000,000.

*Received by the Institute Nov. 13, 1940.
†Engineer of Design, Chicago Department of Subways and Superhighways.

The Milwaukee-Lake-Dearborn Subway is from a connection with the structure of the Logan-Humboldt branch of the Metropolitan Division of the elevated near Paulina Street; thence via Milwaukee Avenue, Lake Street and Dearborn Street to Van Buren Street. The length of this subway is 3.3 miles. The estimated cost is $20,000,000. These are both to be two-track subways for rapid transit train operation.

The long needed initial system had been talked about for forty years—so long, in fact, that there were many persons who said it would never be built. There were others, too, including members of the engineering profession, who cast doubt on the feasibility of constructing a subway in Chicago, particularly in the downtown area, because of the unstable nature of the soil, the interferences with existing tunnels and utility net works and the heavy pressure due to tall buildings.

Under the original grant agreement with the PWA, a period of only 18½ months was allowed for substantial completion of the project. This time limit was later extended about 6 months.

A competent staff was speedily organized. The Department was fortunate in obtaining the services of a number of engineers and draftsmen who had just completed a large sewer construction program for the Chicago Sanitary District, and were experienced in tunnel design and construction.

Under the guidance of Commissioner Philip Harrington and Ralph H. Burke, Chief Subway Engineer, the staff quickly adapted all previous local tunnel experience and data to the proposed subway, making whatever modifications in design, construction and procedure were required.

At the time of the approval of this grant, preliminary locations had been made for both the route of the subway and the location of the stations. These two items were checked over very thoroughly and final location determined for both. Detailed maps of the two routes were drawn up and studies made for the location of emergency exits, fan chambers and ventilation chambers, pump rooms for the drainage system and electrical service rooms, cable shafts and splicing chambers.

A 9 ft. 6 in. width of subway car had been agreed upon and clearance drawings made and train tube dimensions established. With this information approved, a very detailed study of the entire route was necessary to determine whether these structures could be built at the points chosen, due to their possible interference with surface conditions, underground utilities, or existing freight tunnels.

During the time that these general items of the design were being covered, actual contract plans for sections of the subway tubes were being prepared, and bids taken for their construction, as the general plan of construction was based on completing all tunnel work first, then proceeding with mezzanine station construction, next connections to the existing Rapid Transit Elevated System. The design followed this construction procedure, and at this time, 84 per cent of the tunnel work and 25 per cent of the station work is completed.

Incline structures to connect with the elevated have not been started, but the bids for this work will be received early in 1941. The system will then be ready for the installation of architectural finish, escalators, mechanical equipment, lighting, plumbing, ventilation equipment, road-bed, track, signals and cars.

Before the actual structural design of the structures could be made, problems of stations, junctions, ventilation, drainage, fire protection, moving stairways, etc., had to be studied. A brief description of these features of the system are given below.

Stations

Most of the stations are of the mezzanine type—below the utilities located directly under the street surface, but above the level of the subway train tubes. Entrance stairs lead from the street surfaces to this mezzanine station, 18 ft. below. On this level are cashier's booths, turnstiles, toilet rooms, concession stands and electrical or other general service rooms.

After paying a fare, the passenger may descend on the escalator, or stairs, to the 500 ft. long loading platform and trains. On this level, 22 feet below the mezzanine are additional service rooms. Four feet below the platforms are the tracks and third rail. This low level system will allow the construction of tunnels above them for street cars.

Each tunnel is large enough for one track only. Loading platforms are of two types: Side platforms built on the side of each tube, or a center platform between the two tubes. Crossovers from one track to the other are needed for emergency operation, and this requires that both tubes be brought together into a single wide tube at these points. The two crossings under the river required special design, as conditions allowed a suitable depth for tunnel work on the Dearborn Street route, but not on the State Street route, and in this case the tubes were fabricated of steel and concrete in a dry dock, floated to the site, and sunk in a dredged trench in the river bed.

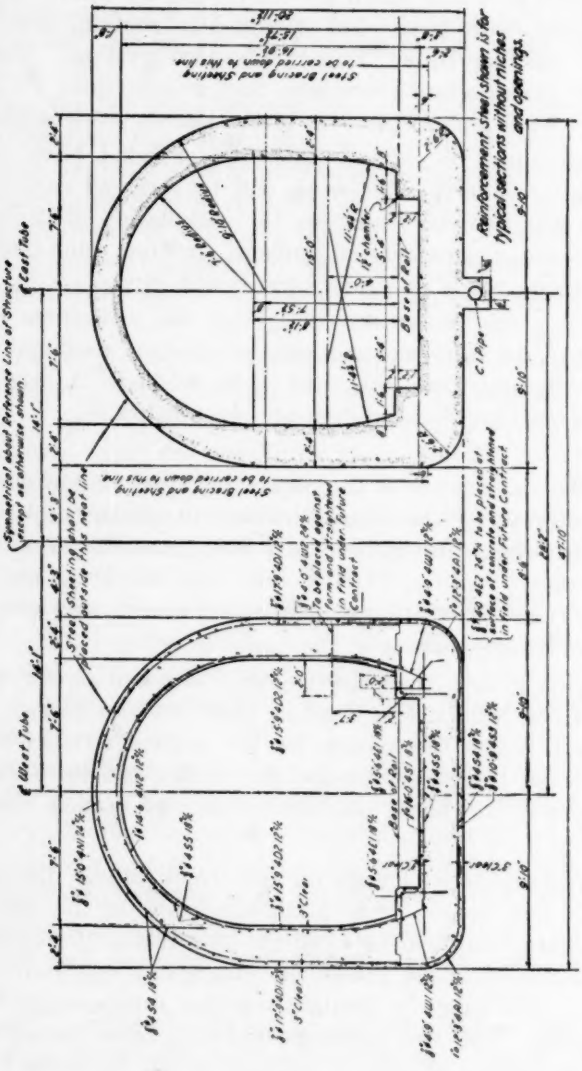


FIG. 1—TYPICAL SINGLE TUBE SECTIONS

Junctions

Special design work was required at the junction of the Lake Street Line and Milwaukee Avenue Line to eliminate a grade crossing. At this point one tube passes over the other. This condition also occurs at 13th Street on the State Street Line.

Ventilation

Ventilation shafts to the surface on about 450 ft. centers, piston action of the train, large fans for emergency use in case of fire, and small fans for the ventilation of some of the stations and service rooms, will be essential items in the ventilation of the entire subway.

Drainage

Estimates were made of the amount of water that will enter the subway by seepage, by rainfall entering portals, stairwells and ventilation shafts and emergency floods due to flooding of the street surface caused by water main breakage or large fires in the vicinity. Pumps have been located along the subway to handle this estimated amount. The larger pumps are naturally located adjoining the crossings under the Chicago River where the subway is at the greatest depth.

Fire Protection

In case of fire, station stairs, emergency exits and ventilation shafts will be depended upon for the entrance of firemen to the tunnel. In the deep tunnel sections at river crossings hose lines will be installed with valves 300 ft. apart.

Moving Stairways

The depth of the station platforms below the surface makes the use of some mechanical means of passenger conveyance a necessity. All provisions are being made in the subway station structures for the installation of 42 moving stairways.

STRUCTURAL DESIGN

The design of all structures was influenced a great deal by the method of construction, whether tunnel or open cut. The single tube sections (Fig. 1), outside of the Chicago Loop are of horse shoe shape with internal dimensions of 15 ft. wide and 16 ft. 9½ in. high. There are 21,950 lin. ft. of this section. On curves where additional width is required for clearance of car overhang beyond the trucks, the internal width varies from 15 ft. to 17 ft. with corresponding slight increases in the wall thickness. There are 2,925 lin. ft. of these sections.

On the State Street route north of the river there are 8,430 lin. ft. of twin tube section, Fig. 2.

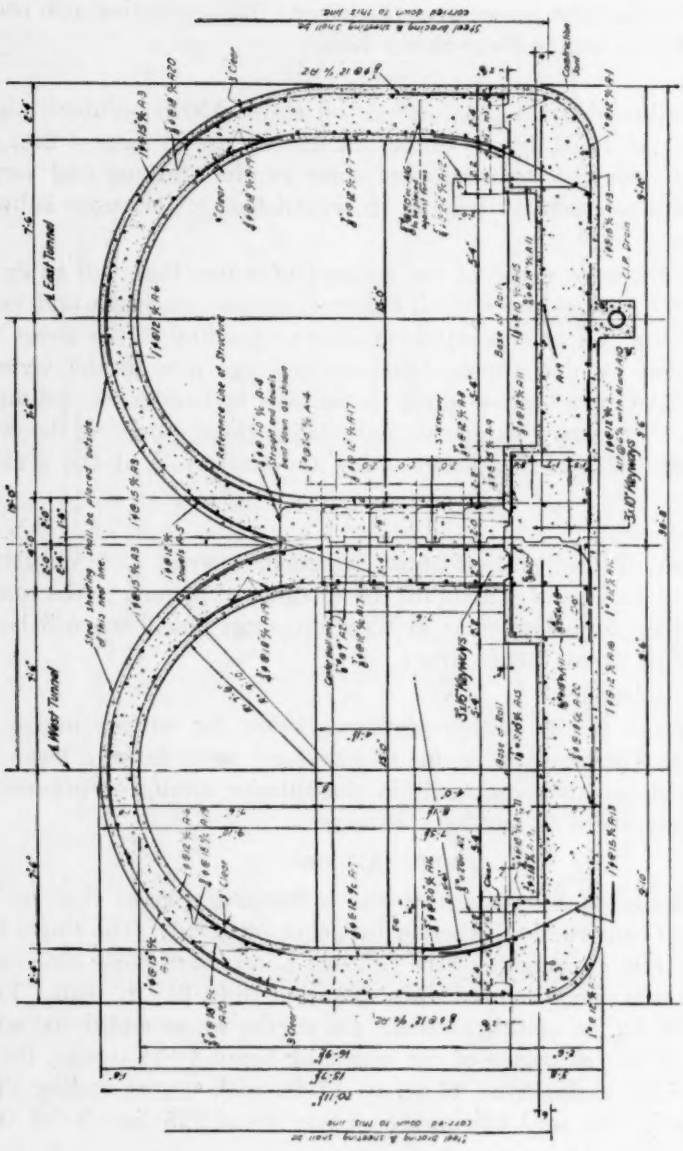


FIG. 2—THERE ARE 8430 LINEAR FEET OF TWIN TUBE SECTION

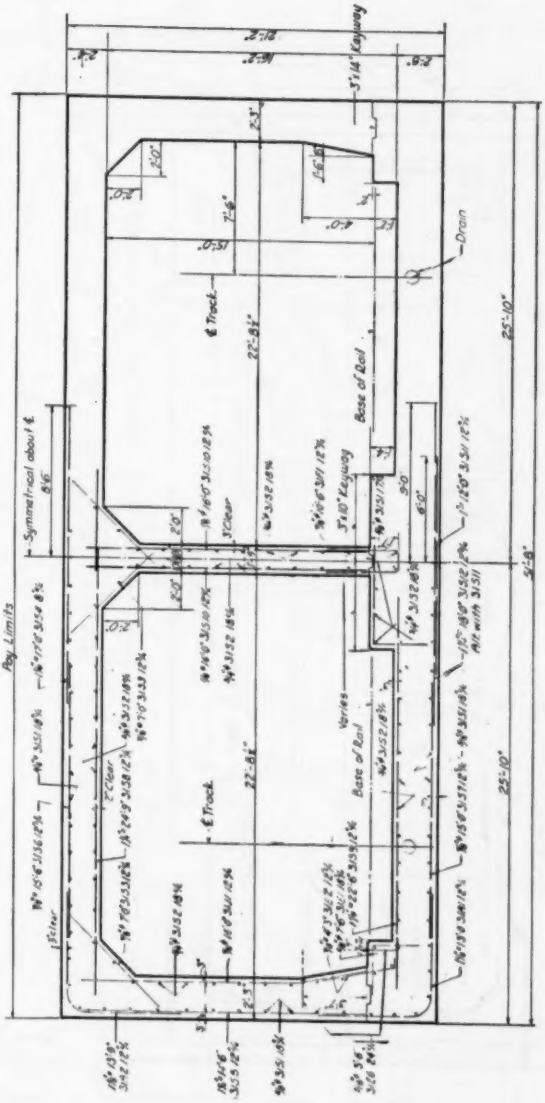


FIG. 3.—SECTION OF TUNNEL IN OPEN-CUT CONSTRUCTION

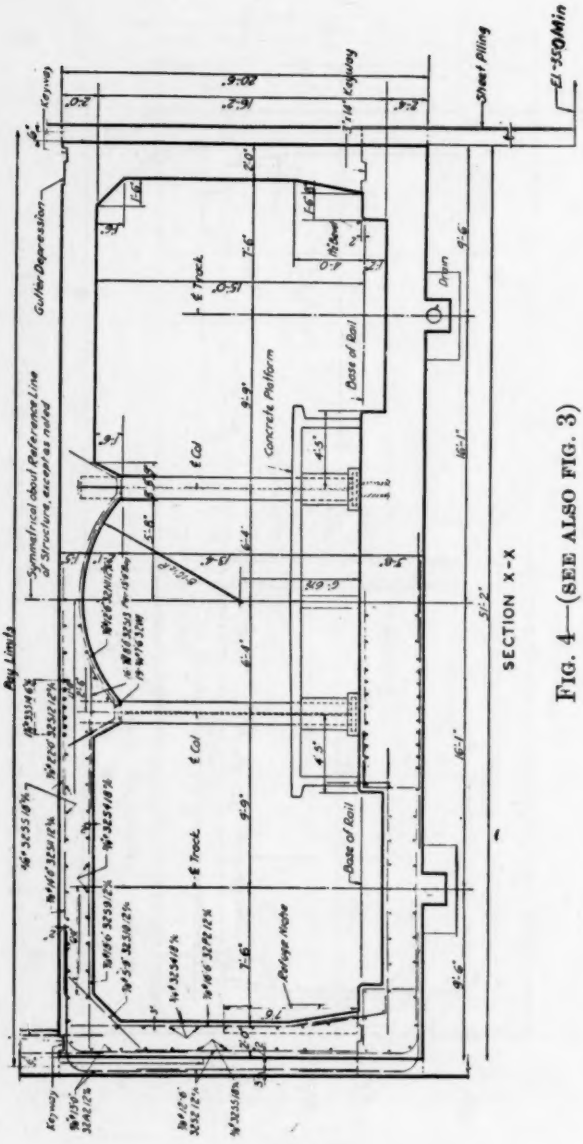


FIG. 4—(SEE ALSO FIG. 3)

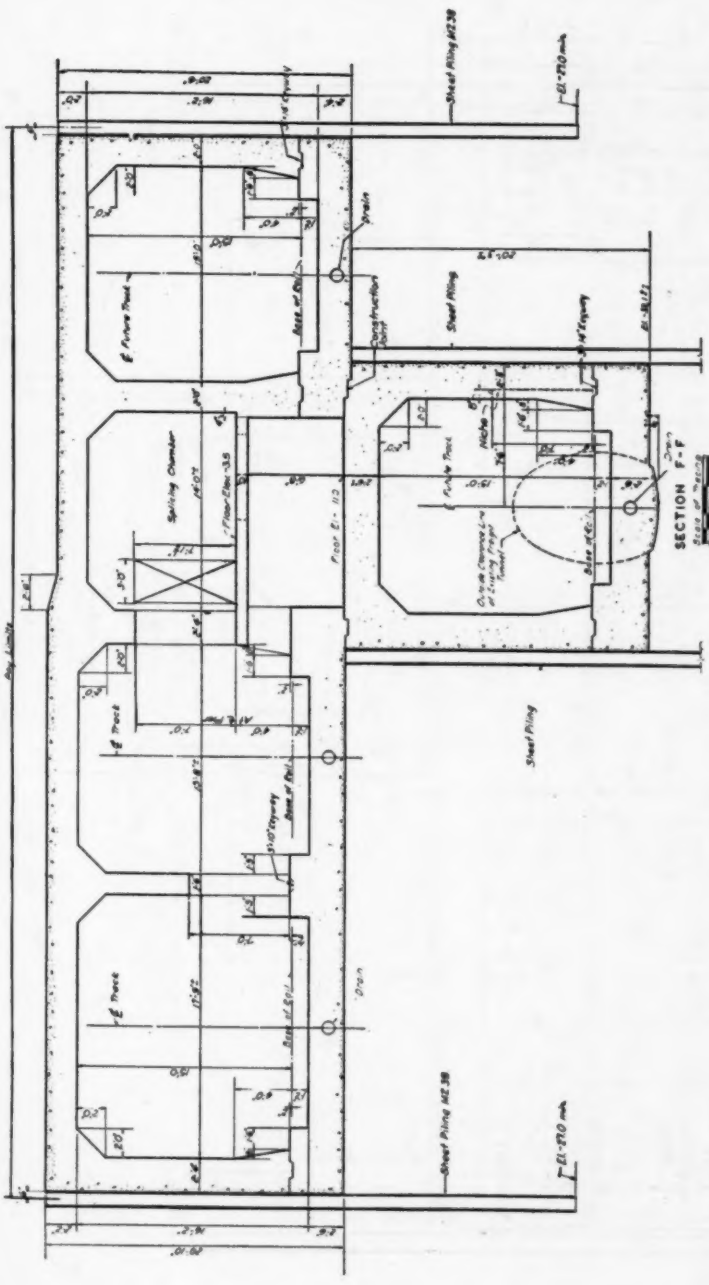


FIG. 5—SECTION IN WHICH SUBWAY BECOMES A TWO-STORY STRUCTURE PROVIDING FOR FUTURE EXTENSION—SEE ALSO FIG. 6

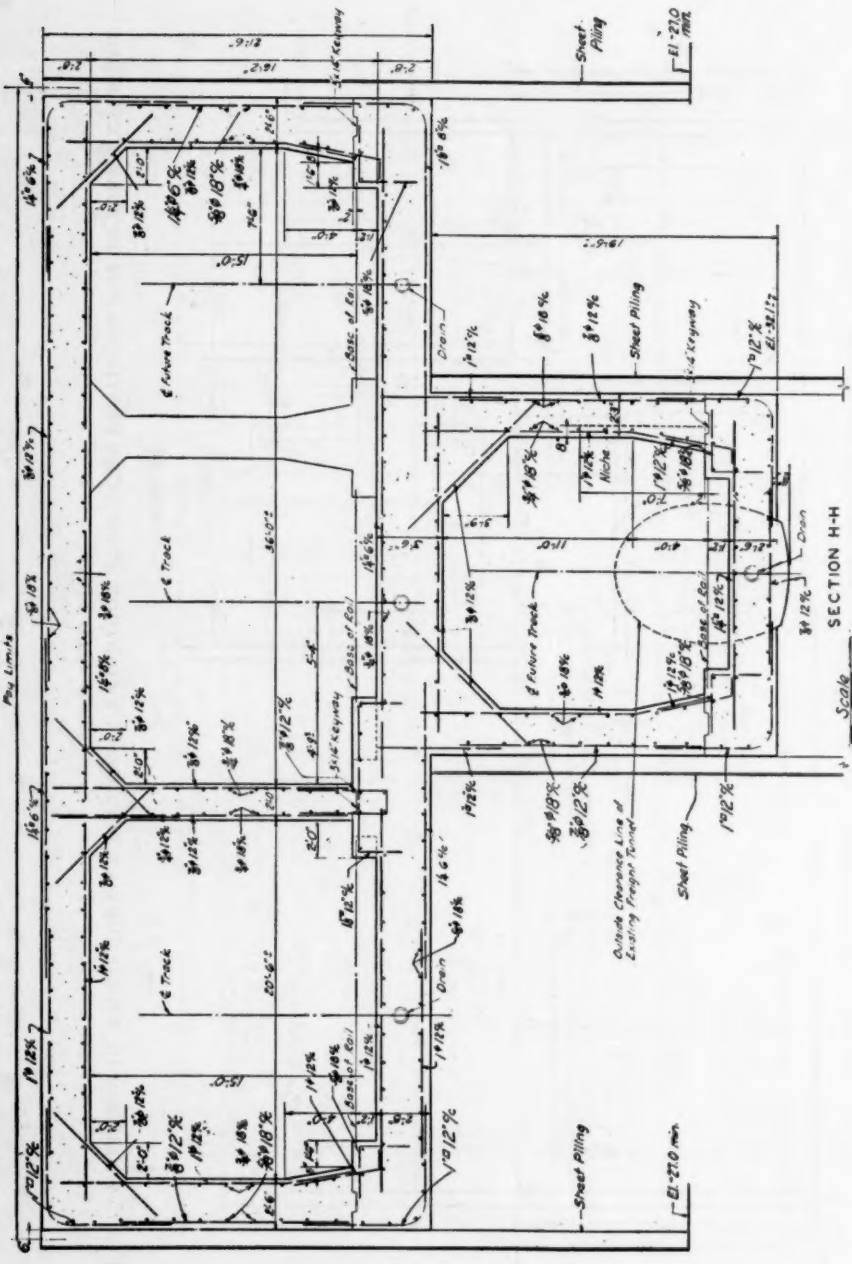


FIG. 6—A SECTION OF TWO-STORY CONSTRUCTION (SEE ALSO FIG. 5)

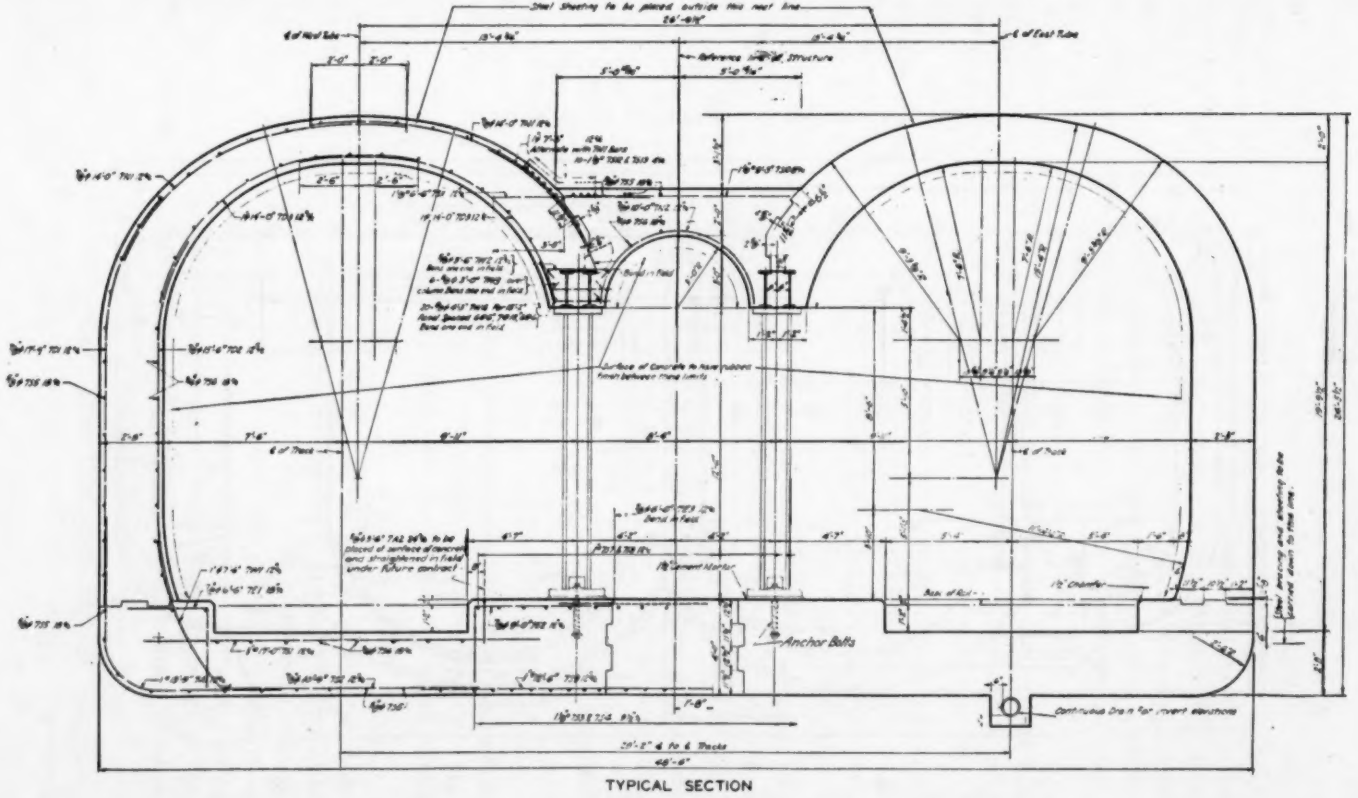


FIG. 7—TWO PARALLEL SINGLE TUBES WITH PLATFORM BETWEEN

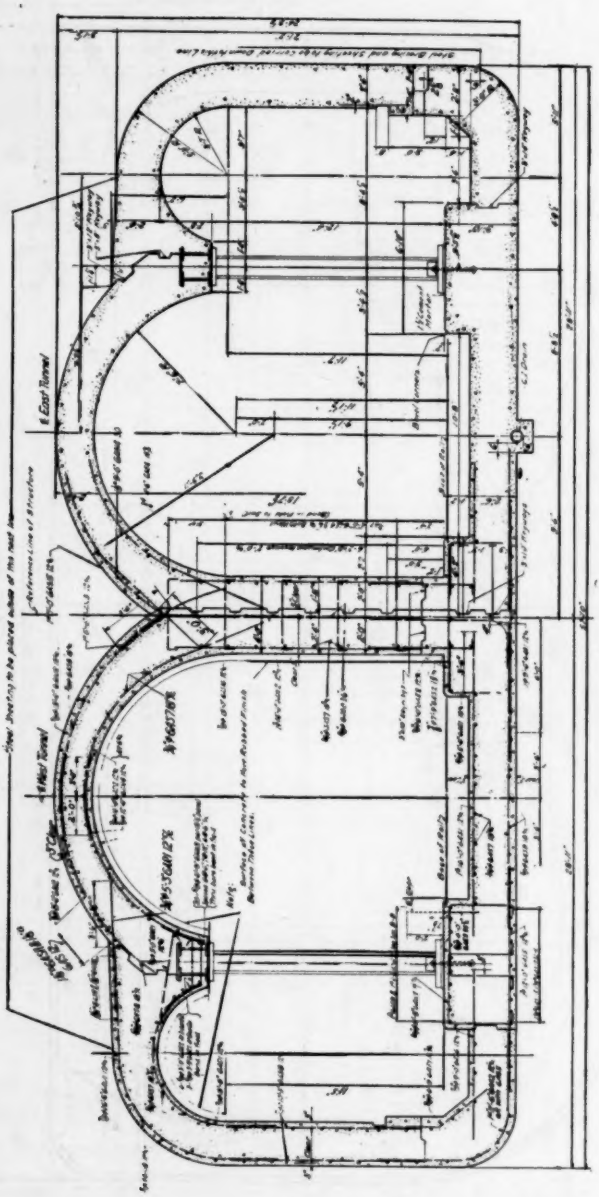


FIG. 8—IN TWIN TUBE SECTIONS PLATFORMS ARE OUTSIDE

Where the overhead cover over the tubes is insufficient for tunneling operations and open cut construction is being employed, single and double box, or rigid frame, structures are used (Fig. 3 and 4). These conditions occur at the ends of the subway routes where they emerge from the ground and connect to the present elevated structures. At the south end of the State Street route the rigid frame section becomes a two-story structure with the low level track providing facilities for future subway extension on south State street. Fig. 5 and 6).

In two parallel single tubes where stations occur, the station platforms are between the tubes. The internal dimensions of the tubes in this portion are 16 ft. 3 in. wide by 19 ft. 9½ in. high (Fig. 7).

In the twin tube sections the station platforms were placed on the outside of the tubes. In this portion the internal dimensions of the tubes are 15 ft. wide by 19 ft. 9½ in. high. The internal dimensions of this arched addition are 7 ft. wide by 16 ft. 4 in. high (Fig. 8).

In the entire system there are three crossovers. Two of them were built by open cut methods (Fig. 9), and one was built by mining (Fig. 10). The length of these crossovers is approximately 180 ft.

At the south end of the State Street route provision is made for future extension of two tubes south. For this purpose the south bound tube is stub ended a short distance beyond the junction with the tubes leading to, and connecting with, the present elevated structure. The north bound tube is also stub ended but at a sufficiently lower level to pass under the present track tubes. Proceeding north at a 3 per cent grade it emerges and reaches the normal level between these tubes. By means of two turnouts, track connections are made with the south bound west track, and the north bound east track (Fig. 5).

On the Dearborn Street route at Milwaukee Avenue and Lake Street, provisions are made for extending the subway west on Lake Street. The inbound or south tube is kept at the average subway level and makes a Y-junction at the street intersection. One arm of the "Y" going up Milwaukee Avenue and the other arm stub ending on Lake Street. The outbound or north tube, as it approaches this street intersection, joins a similar Y-junction but at a lower level so that the Lake Street arm passes under the south tube.

It should be noted in these two cases that at no time do the tracks of trains running in opposite directions cross each other at the same grade elevation.

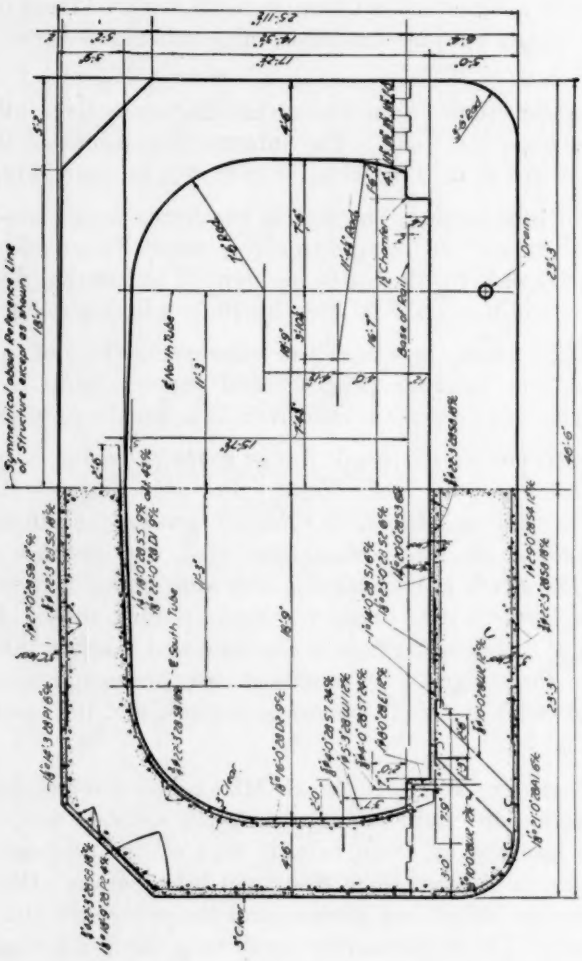


FIG. 9—A SECTION OF "CROSS-OVER," BUILT IN OPEN CUT

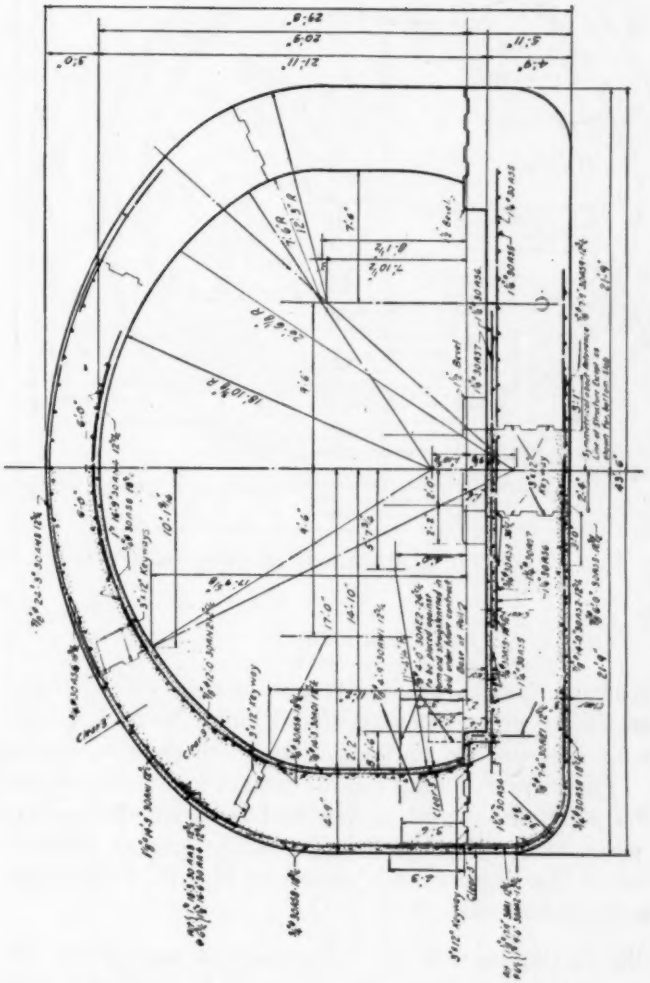


FIG. 10—A CROSS-OVER SECTION BUILT BY MINING

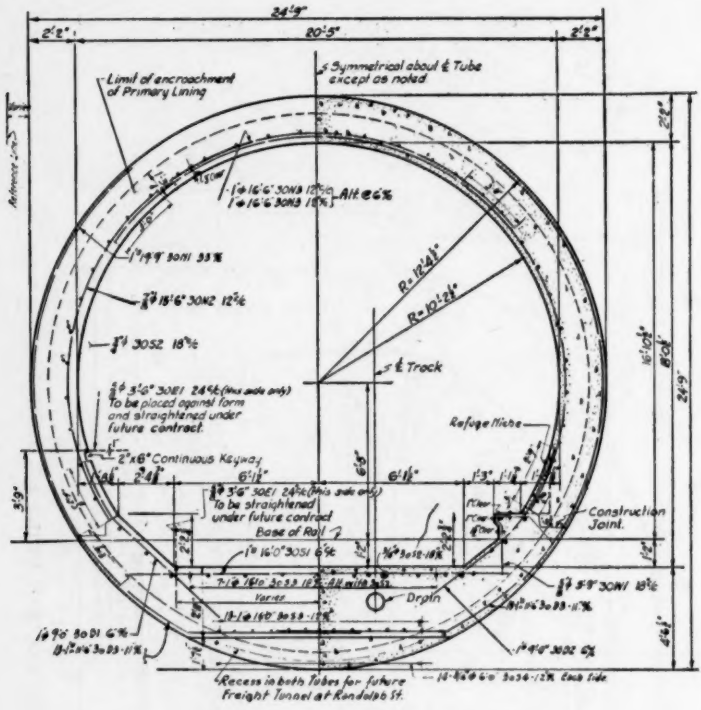


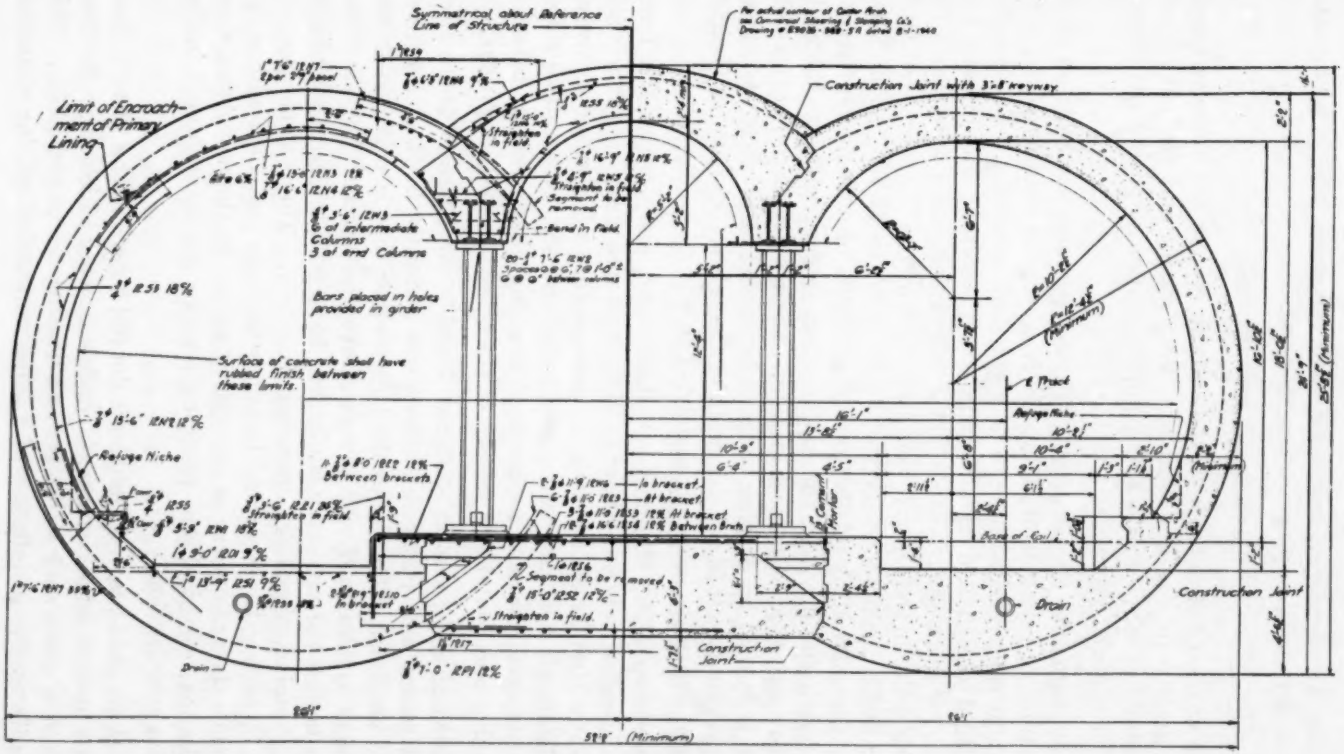
FIG. 11—TYPICAL SINGLE TUBE CROSS-SECTION

FOUNDATIONS

In the Chicago Loop, due to soft plastic clay with a high water content and due to greater hazard of tall buildings on insecure foundations along the subway route, the shield method of mining was employed. The primary lining for the tunnel excavation is made up of structural steel plates and shapes and is capable of carrying the full earth pressure at a steel unit stress of 24,000 p.s.i. The typical cross section of the single tube is shown in Fig. 11. There are 6400 lin. ft. of single tube section.

Within the station section the center portion was mined out after the concrete lining in each single tube had been partly poured in place (Fig. 12). There are 6880 lin. ft. of station section.

Because of the close proximity of station platforms, the State Street river crossing could not be built by tunnel methods without producing excessive grades for train operation. To overcome this


Symmetrical about Reference Line of Structure
Limit of Encroachment of Primary Lining
Construction Joint with 3'-8" keyway
Surface of concrete shall have rubbed finish between these limits.
Bars placed in holes provided in girder
Refuge Niche
Drain
Construction Joint
Segment to be removed
Straighten in field
Bend in field
Between brackets
Between brackets
Between brackets

difficulty a water-tight, twin tube, steel structure, 200 feet long was built in dry dock, as previously mentioned.

AUXILIARY STRUCTURES

The auxiliary structures on the subway system are numerous and vary greatly in size and outline: Vent shafts and fan chambers; emergency exits; electrical and pumping stations; splicing chambers; transfer tunnels; pedestrian passageways.

Vent shafts and fan chambers.—These structures are built on each train tube, at intervals of about 450 ft. along the subway route, and generally vary from a three to an eight shaft structure. To simplify structural design and to eliminate costly sheeting and bracing required for rectangular shapes, these individual shafts were generally made circular in section and were excavated by the "Caisson Method." Their inside diameters vary from 5 ft. 9 in. to 10 ft. About 8 ft. below the sidewalk level the tops of all shafts of a single vent structure terminate in a box, the top of which is flush with the sidewalk and covered over with steel grating. Their bottoms are also a box which is formed by drifting laterally by tunnel methods to the openings previously left in the side walls of the tubes. The average height of these structures is approximately 41 ft.

Emergency exits.—Throughout the system emergency exits have been installed. Where they connect with one tube only, they are generally circular shafts with an arrangement of circular stairways and landings, such as is often used in lighthouses. Where two tubes are connected, the exits are designed as a single box structure between the tubes and containing a series of stair flights and passageways.

Electrical and pumping stations.—Outside the Chicago Loop, electrical rooms and pumping stations are generally built midway between stations. The pumps discharge water that is collected in the drainage system into city sewers above the tubes. The electrical room houses the necessary equipment to distribute current for lighting, pumping, signaling, operating fans, etc. These structures are built in open cut. In several locations they are combined with stairs and thus they also serve as emergency exits. In the downtown area, ventilation, electrical and pumping stations are incorporated in the station construction.

Splicing chambers.—Throughout the full length of the system there will be in each tube, continuous conduit benches which serve the dual purpose of emergency walkways and housings for power cables. This necessitated the installation of a large number of splicing chambers

which were placed generally every three to four hundred feet. In twin tubes they were at the outside walls and in parallel single tubes, between them. They were all constructed by mining methods from the tubes. In the Loop stations they were under the loading platforms.

Transfer tunnels.—To facilitate the transfer of passengers in the downtown area from one tunnel route to the other, two transfer tunnels are being built. These are horse-shoe shaped, with inside dimensions 12 ft. wide and 11 ft. 3 in. high. They connect with the loading platforms.

Pedestrian passageways.—At two locations in the loop pedestrian passageways will connect the station mezzanines opposite each other in the two routes. They are box structures built in open cut with internal dimensions of 12 ft. 2 in. wide and 8 ft. 1½ in. high.

MEZZANINE STATIONS

The subway system has 24 mezzanine stations and three auxiliary pay stations, of which 15 mezzanine and one auxiliary station are within the Loop district. All mezzanine stations excepting one, rest directly on the tubes. Outside the Loop area the stations are at the street intersections. Within the Loop they are in the middle of the block with a stair entrance at each corner of the mezzanine. This brings the stair entrances at the sidewalk at about the quarter points of the block, keeping them away from the heavy pedestrian traffic at the street intersections. The mezzanine floors are connected with the loading platforms in the tubes by means of stationary and moving stairways.

These structures do not present any great or unusual structural problems. They are built in open cut, the contractor having to support all utilities in the street and maintain street car and vehicle traffic by some means of support and decking.

Rehabilitation of Freight Tunnel

The Chicago Loop and immediate vicinity is honeycombed with a freight tunnel system having, in general, a single tunnel on the center line of each street and at about the same elevation as the subway invert. In those streets where the subway is being built, the tunnels are destroyed as well as all transverse crossings. To rehabilitate this system within the limits of the subway, four new transverse connecting links are being built. These links are approximately three blocks long and with only one exception, they have been built sloped toward the subway at a 3 per cent grade, so as to pass under

the subway tubes. In the other case the freight tunnel passes over the subway.

Where the subway emerges from the ground, inclines are to be constructed consisting partly of a fill embankment and partly of a steel trestle connecting with the present elevated structures. In this vicinity the elevated tracks need to be spread laterally to permit the subway incline to come up between them. This requires not only extensive alterations, but long sections of new structure, with new columns and new foundations for the existing as well as for the new connecting structure. All of this work will have to be done without interruption to elevated traffic.

DESIGN OF TRACK TUNNEL SECTIONS

The tunnel sections encountered in the design of the subway may be conveniently divided into two general types, viz., arch sections, comprising single and multiple tubes of either horseshoe or circular shape, and box or rigid frame sections, comprising single frames and one-story and two-story multiple frames. The various and constantly varying requirements of external loads, soil conditions, track clearances and alignment, nearness of existing structures, openings for, and connections to, auxiliary subway structures, etc., precluded the use of standardized sections with fixed dimensions and, hence, it was necessary to analyze and design a number of tunnel sections differing not only in arrangement but also in shape and size.

Loading conditions and resulting pressures against the tunnel walls were not at all uniform. The ratio of lateral pressures to vertical loads is indeterminate for Chicago clay varieties, and cannot be established by laboratory experiments. It is, as is well known, a function of the physical and chemical qualities of the clay, of depth location, shape and rigidity of tunnel, grouting materials, method and speed of grouting and, above all, of time. In fact then, the lateral pressure on the tunnel tubes varies within very wide limits and 1 to 2 years hence this pressure may be twice as great.

It follows that the tunnel ring stresses vary approximately to the same extent so that the stresses at some point may now be compressive, vanish in 6 months and reappear as tensile stresses in another 6 months. In addition, we must consider the stress variations and reversals resulting from construction operations, from the load effects of contiguous existing structures, proximity of parallel tunnel tubes and sudden, severe changes in section and load distribution due to connections to mezzanine stations and auxiliary structures, viz., ventilating shafts, pump rooms and fan chambers.

These unbalanced loading conditions occurred singly and in combination, and for a distance of about two track miles, the lateral pressure against one side of the tunnel exceeded that against the other by from 10 to 100 per cent.

To maintain the necessary internal clearances and to provide continuous tunnel sections of fairly uniform shape and dimensions it was, obviously, not feasible to take full cognizance of all the local conditions that affected the design. After a prolonged study of the pertinent factors, it was decided to adopt the following design assumptions:

1. The top pressure to act vertically; to be distributed uniformly, and to vary directly in intensity with the depth of cover and weight of arch.

2. The base pressure to act vertically; to be distributed either uniformly, or trapezoidally in accordance with the magnitude and the location of superimposed concentrated loads, if such were present, and to be equal to the top load plus weight of arch ring.

3. Lateral pressure to act horizontally; to be distributed trapezoidally and to have a magnitude of either one-third or two-thirds of the vertical pressure intensity at the point of application.

These stipulated design assumptions were considered adequate to take care of most of the minor local variations in lateral pressure intensities. Consequently, all normal arch rings with ordinary normal loadings were analyzed and designed for two conditions, viz., (1) Full vertical load and a lateral load of one-third of the vertical load, on both sides, and (2) Full vertical load and a lateral load of two-thirds of the vertical load, on both sides. A number of ring sections were designed for unsymmetric loadings, that is full vertical load and a lateral load of one-third the vertical on one side and a lateral load of two-thirds the vertical, on the other side.

Superimposed concentrated loads and other loads applied eccentrically or unsymmetrically were considered separately. The portion of the platform sections located directly under, and supporting the mezzanine stations demanded a number of unusual design procedures. Extra heavy load concentrations on the arches, 11 ft. by 30 ft. openings in the crown for stairways and escalators, 6½ ft. by 21 ft. openings in the base slab for passageway connections, cable splicing facilities and other operating features, gave rise to numerous intricate and difficult problems.

The assumptions as to pressures and pressure distribution used in the analyses have been supported by actual field tests.

Column loads and deflections were accurately determined by field measurements for a total of 34 station columns and it was found that the average differences between assumed and computed loads and deflections and the corresponding measured actual values, did not exceed ten per cent. One of the causes for this discrepancy may be the fact that the earth pressures against the tubes do not act vertically and horizontally as assumed in the calculations. The arching effect of the undisturbed clay above the tubes is another contributing factor which will, probably, continue to exist almost indefinitely unless disturbed by future excavations nearby.

Practically all arch sections were analyzed for stresses by the classical Methods of the Theory of Elasticity. They are known to be mathematically exact and, with the proper simplifications and suitable arrangement of calculations, they may be used with great rapidity in the analysis of any tunnel section, whether circular or horseshoe, single arch or multiple arch, symmetric or unsymmetric and with either constant or varying moments of inertia. Moreover, the individual steps may be recorded in a manner to permit a constant check of the calculations and to facilitate the subsequent determination of shears, thrusts and arch deformations.

These methods require, as is well known, the solution of as many simultaneous equations as there are redundants. The arch tunnel sections contained from 2-5 redundants, but by the use of polar-symmetric loadings, it was rarely necessary, even in the case of unsymmetric loadings, to integrate over the entire section, or to solve for more than four unknowns at any one time.

For purposes of stress analysis the axis of the frame was taken as the locus of the center of gravity of each arch section and the moments of inertia were calculated for the full concrete section about this axis.

The rigid frame sections required an entirely different analytical approach, even though the method of elastic weights would still be applicable. Some of these sections were highly indeterminate, containing as many as eleven redundants. In the analysis of the single-story frame sections, a modification of the three-moment equation was used. As the frames consisted mainly of haunched members it was necessary to express the end moments in terms of end rotations instead of span lengths and moments of inertia, as is customary in the case of prismatic beams.

This form of analysis, like the method of elastic weights, is mathematically correct, and can be used very conveniently and effectively for structures with as many as eight unknowns, provided such structures suffer no displacement of the joints.

The necessary equations can be arranged in such manner that their solution requires rarely more than one sheet of calculations, regardless of the number of equations. Furthermore, the simplicity of the arrangement is such that men with very moderate mathematical abilities find little difficulty in solving these equations.

Two-story multiple frame sections are more formidable to analyze, since the degree of indeterminacy increases with the complexity of the structure. This condition, coupled with the fact that one or more joints were subject to axial displacement, made such structures indeterminate externally as well as internally. Because of these complications, the method of moment distribution was used as the quicker and the more suitable scheme of analysis.

Some of the heavier arch sections of wider spans were investigated for temperature stresses and stresses resulting from the effect of axial thrusts and shears. The analysis showed that the thermal and rib shortening stresses were so small in comparison with the axial and flexural stresses resulting from the applied loads, as to be practically negligible and, consequently, they were omitted from the final calculations.

The specifications of the Department of Subways and Superhighways for maximum allowable unit stresses in concrete and steel conform with the A. C. I. Building Code. (Serial Designation 501—36-T., Sections 305 and 306.)

As the construction and design of these tunnels were so closely interwoven, some detailed features of the design are necessarily included in another paper* on construction of the subways.

*Construction of Chicago's Initial System of Subways," by V. E. Gunlock, in this JOURNAL.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL Supplement concluding this Proceedings volume

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Construction of Chicago's Initial System of Subways*

By V. E. Gunlock†

SYNOPSIS

The construction of subways in Chicago by tunneling methods has been a moot question for many years. Proof that this can be accomplished now exists in the work now being completed. These subways have been constructed by tunneling methods through Chicago's underlying stratum of soft blue clay. The paper describes the two methods used in the subway tunnel construction; mentions some of the special problems encountered; describes methods of placing the concrete, and explains the control exercised to insure sound concrete in the finished structure.

The Chicago subway is a low level structure with the top of train tubes about 25 ft. below the street surface. The train tubes and the station platforms are built in tunnel prior to the construction of mezzanine stations and auxiliary structures.

SOIL STUDIES

At the beginning it was necessary to make an extensive study of the soil and to obtain data as to buildings and foundations along the line. Some test pits were sunk but most of the data were obtained from borings placed at intervals of about 300 ft. These borings were made by forcing Shelby tubes into the soil.

Generally speaking, the subsoil along the two routes of the initial system was a soft plastic clay with large variations in plasticity and water content. Along some sections, the water content of the soil was found to be as high as 58 per cent. Sand pockets were also encountered.

*Received by the Institute Nov. 13, 1940.
†Senior Assistant Subway Engineer, Chicago Department of Subways and Superhighways.

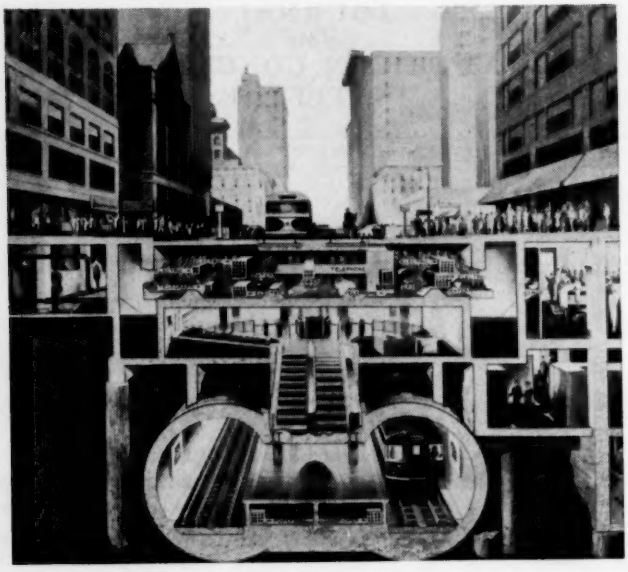

FIG. 1—FROM SCALE DRAWING SHOWING CROSS-SECTION DEARBORN STREET SUBWAY

EXCAVATION

The tunnel portions of the subway outside the loop were of a horseshoe design and comprised approximately 75 per cent of the entire route. These portions were excavated by hand methods using I-beam ribs and steel plates for support of the earth during construction. The work was done under air pressure of ten to fifteen pounds per square inch, and the concrete arch was placed as quickly after the tunnel was mined as construction methods would allow.

The earth was loosened by means of power knives, hand knives or air spades according to the consistency of the earth, but the great preponderance was loosened by power knives motivated by cables and air hoists.

The mining was carried out with a series of benches or levels, allowing efficient working space for many men in the heading. The method found most successful was called the wall beam method, illustrated in Fig. 2. The first step was the mining of the pilot drifts, 1 in Fig. 2, which were three feet wide, four feet high and were carried twelve feet ahead of the section marked 2. Wall beams of eight-inch H-beam sections were placed with horizontal web in the pilot drifts on wood foot blocks. Section 2 was excavated and steel arch ribs

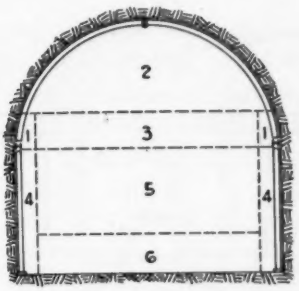

FIG. 2—SHOWING STEPS IN WALL BEAM METHOD OF TUNNEL LINING—SEE TEXT

set to bear on the wall beams. Liner plates were placed and section 3 was excavated.

After the sides (marked 4) of the lower benches were mined, the lining of the arch was prestressed and forced against the roof by raising the wall beams with hydraulic jacks bearing on foot blocks on the invert. I-beam legs were set on two or more layers of wood block, wedges were driven between these blocks and the jacks were removed. Horizontal wood struts with screw-jack ends were inserted between the wall beams every six or eight feet to take horizontal thrust and to prestress the lining in a horizontal direction.

Section 5 was then excavated, but section 6 was left as bracing for the temporary lining and excavated just ahead of the casting of the invert concrete. This procedure was used on nearly all contracts for the horseshoe type tunnel, and its advantages were: (1) Minimum number of transfers of load; (2) Simplicity of steel lining; (3) Ease of prestressing the lining; (4) Minimum of interference with placing of concrete lining.

The work generally progressed in accordance with a 48-hour cycle, of which 32 hours were devoted to mining and 16 hours to concreting. In places where the earth was of poorer quality, it was thought desirable to introduce a variation in this scheme by concreting the invert every 24 hours to provide more positive horizontal bracing between the bases of the wall columns. Concrete arches were placed at 48-hour intervals in all cases.

As the hand mining progressed it became more and more evident that this method would not be suitable for the loop district. The shield method of mining was adopted for this portion and the principal reasons for this were as follows:

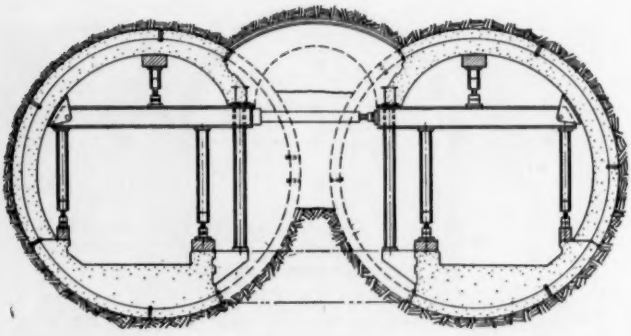

FIG. 3—TYPICAL PROCEDURE IN BUILDING TUNNEL IN CHICAGO LOOP DISTRICT

1. The uncertainty of holding 10 to 15 pounds air pressure in areas where there is a maze of underground utilities, as well as numerous structures, such as subbasements that extend 20 ft. or more below the street surface.

2. Higher water content of the clay lowered the reasonable and safe bearing values of the soil for carrying the steel lining and the loads it had to support and made a full circular steel lining desirable.

3. The probability that the lighter steel lining permissible in the hand mining method would not withstand the pressures during the intervals between concreting, especially if the air pressure should be lost or lowered.

4. Ground pressures from the superimposed load of tall buildings on spread footings would require a heavier lining than could conveniently be handled by hand.

5. Progress would be handicapped using the hand mining method because the certainty of soil movements at the face would require an elaborate system of breasting. Furthermore, the hazard of losing a heading with consequent loss of life and property damage would be present at all times.

Events have clearly demonstrated the wisdom of this decision both from speed and safety viewpoints.

SHIELD MINING

The shield design was similar to others used in the past. There were four shields in the loop district. Each shield was 24 ft. 10½ in. inside diameter, 19 ft. 6 in. long, and the weight was 225 tons.

The typical procedure of building the tunnel in the loop district is illustrated in Fig. 3. The primary lining was installed under the protec-

tion of the tail of the shield, after which the invert was concreted as shown. The permanent steel girders and columns were placed and part of the arch was then concreted as indicated. Temporary steel bents were then erected under the steel girders to prevent load from being transmitted to the permanent steel columns until the center section of invert was concreted.

The center section outlined between the tubes was mined, the top lining installed and part of the primary tube lining removed. The center section of the invert was concreted after which the temporary steel bents were removed, allowing the girder loads to be transmitted to the columns. The casting of the center arch completed the concrete lining.

The shield was moved forward by means of 24 hydraulic jacks, each having a capacity of 200 tons at 5000 lb. pressure. The actual pressures required were much less than maximum capacity. As the shields moved forward, the clay squeezed from the cutting edge into the working compartments through ports the area of which was from 5 to 20 per cent of the total face area of the shield. Heaving or sinking of the street level could be controlled by varying the area of the ports. The clay was generally mined in the working compartments.

The ease of operation of the shields was remarkable. For hours at a stretch it was possible to push the shields so closely to line and grade that a five-cent piece would cover the variation from true line and grade. The maximum variation was well within the allowable tolerance and the average deviation was approximately one inch. It was also possible to shove the shield around a curve of 275 ft. radius. Progress on all shield operations averaged 26 ft. each 24 hours.

Temporary horizontal struts capable of taking compression or tension were installed on every other ring for a distance of 50 ft. immediately behind the shield. Similar bracing was erected in the lining of the first tube 50 ft. ahead of the shield in the second tube and left in place until the shield in the second tube had passed.

To minimize settlement the specifications required that voids back of the primary steel lining be filled. With hand mining, irregularities in excavation were unavoidable and with shield mining some clearance had to be allowed between the tail and the steel lining. In both cases, pea gravel was forced into the voids by means of air pressure immediately after the lining was placed.

Shortly afterwards a one-to-one cement grout was forced into the space occupied by the gravel. A compressed air machine was used

operating at about 30 to 35 lb. pressure. The grout solidified the pea gravel and filled remaining voids outside the steel lining.

After the concrete arches were cast, there were voids left between them and the steel lining which also had to be filled. These voids were filled with cement grout injected through pipe sleeves in the concrete arches. Later, when cores were drilled through the arches, the cores revealed a solidified group of layers which from inside to outside were arranged as follows: Concrete arch, grout, steel lining, grout and pea gravel.

CONCRETE

A modified cement specification (C9-38 of the A. S. T. M.) was adopted with a content of not more than eight per cent of tricalcium aluminate compound computed from the finished cement analysis in accordance with the method outlined by R. H. Bogue.*

Admixtures in excess of two per cent in weight were not permitted but, in general, no admixtures were used.

The low aluminate content resulted in a slower initial set which hampered mining operations insofar as more time was required before work could be resumed over newly placed concrete. One contractor overcame this difficulty by using vacuum pads connected with free air. The air pressure in the tunnel caused enough differential to exhaust a large part of the free water from the invert concrete. This shortened lost time by at least two hours per day.

Mixing and Placing

Three different types of concrete mixing plants were used:

1. Paving mixer on street surface discharging through a vertical pipe equipped with a hopper and an airlock. From this the concrete was deposited into either a pumpcrete machine or a pneumatic placer.

2. Paving mixer on surface discharging by means of an inclined boom and bucket into a pumpcrete machine, also on the surface, from which the concrete was pumped direct into the forms.

3. A central mixing plant from which the concrete was hauled by truck to pumpcrete machines in the street and delivered from there direct into the forms.

It was found that about 1000 ft. of pipe was the limit for pumping. In warm weather and when the concrete was in transit in the pipe for about ten minutes, there was from one to two inches of difference in the slump at the machine and at the point of discharge. When the pumpcrete pipe had two bends leading up to and into the top

*Paper 21, Portland Cement Association Fellowship, entitled "Calculation of the Compounds in Portland Cement."

portion of the arch, an air stream under high pressure was introduced into the pipe to assist in moving the concrete.

The conditions under which concrete had to be placed in various parts of the tubes under compressed air were unfavorable at best, so vibration was also used. The vibrators were either external or internal and air jets were also used.

Internal vibrators were employed in the inverts on some contracts, while on others both internal vibrators and air jets were used. The air jets served well in moving the concrete without appreciable segregation. In the arches, the air jets operated by men inside the forms were much more effective than external vibrators in keeping concrete moving on the forms. External vibrators were used on all arch forms.

It was difficult to make the concrete fill spaces around the I-shaped steel ribs in the temporary lining. The best procedure was to keep on discharging concrete through the pipe until its discharge end was well surrounded with concrete. Pumping an additional quantity, say a cubic yard of concrete, through the pipe was found to help in filling pockets. The extreme end section of the pipe was then removed and the operation repeated. Even then it was difficult to fill the forms completely, especially at the crown of the arch. Core drilling showed that subsequent grouting of voids was necessary.

Pipe sleeves were built into the concrete lining and used for grouting voids inadvertently left between the concrete and the steel lining. The grouting operations were supervised carefully and the results were fully satisfactory as revealed by cores taken from the finish lining.

SPECIAL PROBLEMS

Numerous problems arose and were solved by special operations. One of these was the Chicago River crossing for the State Street subway. It was built as a prefabricated tube lowered into an open specially prepared trench in the river bottom. Most of the outer concrete casing was placed in dry dock but the top portion was placed by tremie after the tubes were in place. The ends of the tube were enclosed in coffer-dams built of interlocking Z-shaped steel sheet piling.

There were two special problems in underpinning, one of which involved the Selwyn-Harris Theaters where the Dearborn Street tunnel curves from Dearborn Street into Lake Street. Great care had to be exercised in guiding the shields around the curve since the clearance between it and some of the caissons was as little as nine inches. A similar problem arose where the subway curved from

Division Street into State Street. The problem here was to "pick up" heavily loaded reinforced concrete columns in the twelve-story Will-Mar Hotel. The fireproofing was stripped off to the reinforcement over a vertical distance of thirty-six inches. One T-shaped steel member for each longitudinal bar was provided and notched on the edge of the web so as to avoid touching the spiral. The bottom end of the T-beams was set on a three-inch thick steel bed plate split in two with semicircular holes made to clear the stripped column. One T-beam was welded to each vertical bar but not to the spirals. A one-inch thick steel jacket thirty-six inches high was set around the T-beams on the bed plate. It was welded together and filled with one-to-two mortar. Loads up to 550,000 lb. were lifted successfully.

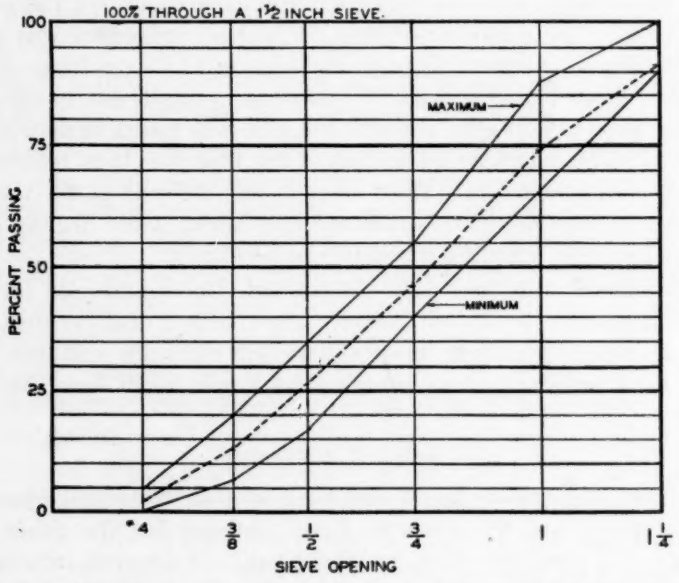

FIG. 4—TYPICAL GRADATION CURVE FOR COARSE AGGREGATE

MATERIALS

When the "Initial System of Subways" is completed there will have been about 2,000,000 cu. yds. of excavation, 135,000 tons of reinforcement and structural steel will have been erected. Other quantities used were 1,000,000 bbl. of cement, 500,000 cu. yd. of stone or gravel, 400,000 cu. yd. of sand, 230 miles of electrical conduits.

All of the concrete going into the subway lining was inspected as to quality and combination of ingredients and mixture. Gravel was obtained entirely from the Valparaiso Moraine of the Wisconsin

glacier. This moraine extends around the south end of Lake Michigan. The deposited material was derived originally from the Niagara limestone which is the bed rock of that territory. Some of the gravel had to be crushed and all of it had to be washed, sized and combined to fit the specification grading. A typical gradation curve of coarse aggregate is shown in Fig. 4.

The crushed stone was all derived from the Niagara formation, and gravel pits as well as stone quarries were closely inspected. Material was rejected at the source rather than at the yards or construction site.

Stock piling of sizes for combination at the plant, as well as at local storage yards, had a tendency to segregate and to control this, frequent sieve analyses were made of samples from the loading conveyor belt at the producing plant, from the output of the batching hoppers at the local yard and at the mixers. Inspectors making these sieve analyses had the duty to direct to some degree the handling of both coarse and fine aggregate. Stock piles were built up in layers to prevent segregation, and sand piles were occasionally worked over with clam shells to restore the original grading.

Gravel was used in most of the underground structures, but crushed stone was preferred to some gravels in structures reaching to the surface because it was more resistant to freezing, thawing and abrasion than were some of the gravels available.

The sand was taken by sand suckers from the "Indiana Shoals" in Lake Michigan. The boats hunted around until they found a source of supply that came as close as possible to the grading required. The sand suckers were equipped with screens which automatically rejected large size grains not allowed in the grading. A typical gradation curve of fine aggregate is shown in Fig. 5.

CONCRETE CONTROL

Most of the concrete materials were batched by weight at the supply yards from which the material was trucked to the mixers in operation. This simplified the control greatly as there might be a dozen or more mixers going simultaneously. One contractor set up his own mixing plant at the yard and trucked ready-mixed concrete to the job.

In order to describe the most important phase of concrete control, reference is made to the concrete proportioning form shown in Fig. 6. The main purpose of this form was to produce uniformity of concrete on all the various jobs and contracts regardless of local variations. To do that, it was necessary to make two corrections, one for lap grading and another for moisture content in the aggregate.

A basic one-bag mix was worked out theoretically and by trial to preserve a workable concrete throughout all the variations to be

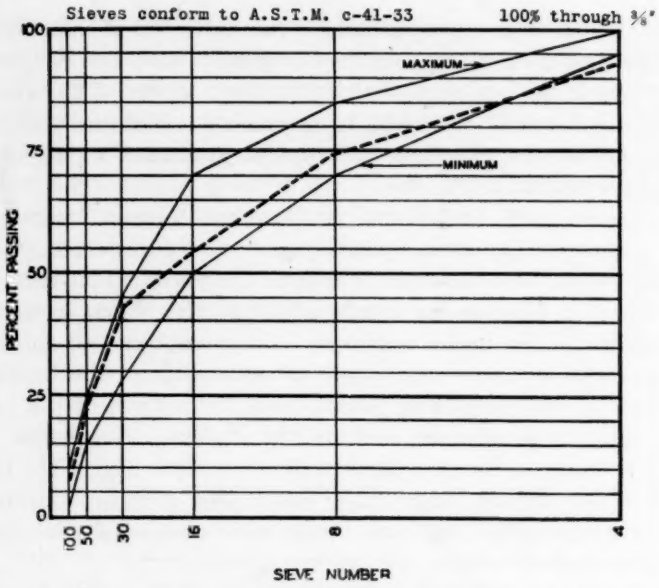

FIG. 5—TYPICAL GRADATION CURVE FOR FINE AGGREGATE

expected in lining the subway tunnels. In this mix a definite division at the No. 4 sieve, was assumed between coarse and fine aggregate. Actually, there was a variation in the amount of sand held on the No. 4 sieve and in the amount of coarse material passing the No. 4 sieve. The correction for overlapping was made in accordance with the formula in column 3 of Fig. 6. The plus sign indicates that the basic mix weight of sand must be increased to have enough sand to give the weight called for by basic mix if the plus No. 4 material is disregarded. The minus sign indicates that the basic mix weight of coarse aggregate must be reduced in order to have just the weight left that is called for in the basic mix.

The per cent of grading correction in column 2 was obtained by sieve analysis and column 5 was obtained by determination of water content. This form was filled out before the beginning of concreting every day and the first truck that left the batching plant carried a copy of the form including the information of maximum amount of water allowed to be added at the mixer.

During the progress of any run of concrete, samples were taken at frequent intervals and the information relayed to the inspector at the mixer.

The general result of this control was reflected, it is felt, in the uniformity of the cylinder tests. Fig. 7 represents a typical record

CONTRACT .....
BASIC CONCRETE MIX No. 4
FIELD CONCRETE MIX No. 213
GROUT MIX No. .....

BATCH MIX AT Material Service Yard No. 1 DATE 10-28-39

Cement	564		
Sand	1723 + 56 = 1779	lbs. Brand Uni.	Source Buffington, Ind.
Coarse Agg.	1817 + 22 = 1839	lbs. (0 to 4) - - -	Source Indiana Shoals, Ind.
Water in Agg. Field	78	lbs. (No. 4 to.....) -	Source Lockport, Ill.
Added Water Allowed Field	247	lbs.	29.64 gals.

FIELD CORRECTION FOR BOTH FREE WATER AND LAP GRADING

	(1) Basic Mix for 1 Bag	(2) Grading Correction	(3) (1) × (2) / 1 − (2)	(4) (1) + (3)	(5) 7:30 A.M. % Free Water	(6) (4) × (5)	(7) Scale Weight (4) + (6)	(8) Batch Wt.
Sand	270	.07	+20.3	287.1	.033	9.47	296.57	1779
Coarse Agg.	320	.01	− 3.2	302.9	.012	3.63	306.53	1839
Totals	590		17.1	590.0		13.10		
W/C	6.5							

	A.M. 11:15	4:00 P.M.	P.M. 9:00	Following Day A.M.
Water in Sand, Mixer.....	54	52	46	lbs.
Water in Coarse Agg., Mixer.....	22	20	23	lbs.
Water Added.....	240	236	246	lbs.
Total Water at Mixer.....	316	308	315	lbs.
Admix.....	12 lb. Fly Ash	12 lb. Fly Ash	12 lb. Fly Ash	lbs.
Slump, Mixer or Tunnel.....	5¼"	5"	5½"	
Temperature of Concrete.....				
Max. Water Added, lbs.....	240, 11:30 a.m.	236	246	
Max. Water per Sack.....	6.32	6.16	6.3	gals.

FIG. 6—FROM "CONCRETE PROPORTIONS" FORM

of cylinder strengths together with water-cement ratio and slump tests. Attention is called to the fact that the strength variations in Fig. 7 are well within 10 per cent of the average. Variations shown can be accounted for to some extent by known differences in water-cement ratios which were varied to suit the type of placement. Contractors insisted upon a more liquid consistency for the concrete going into the arch and the side walls than for the concrete in the invert. The change in water was, of course, kept within the limit set by the specifications.

The lack of "wild" breaks in the cylinder strengths is noteworthy and is attributed to close control and to consistent testing procedure. Referring again to the proportioning form, it is seen that the weight of sand was not much less than that of the coarse aggregate. It is

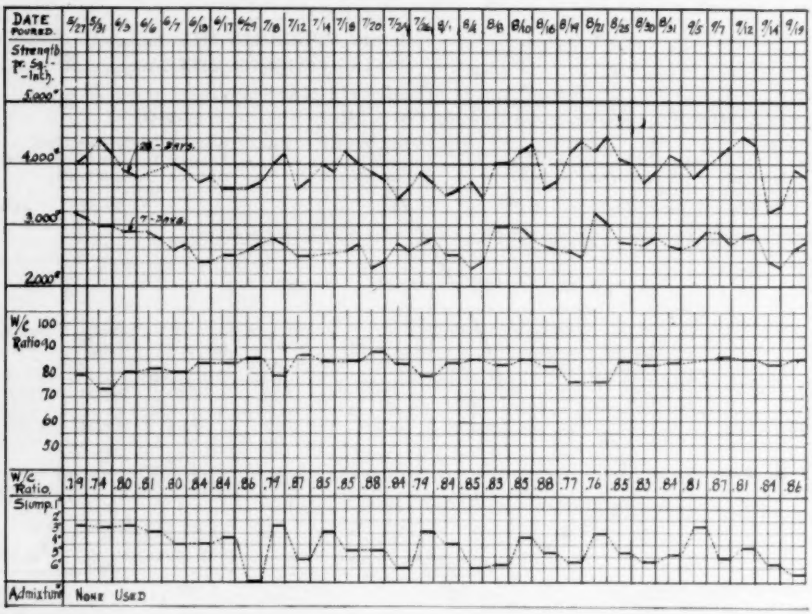

FIG. 7—A TYPICAL RECORD OF CYLINDER STRENGTHS, WATER-CEMENT RATIO AND SLUMP TESTS

believed that this well-sanded mix was desirable because it was more workable and it probably also contributed to the uniformity of the test results. The specifications called for a minimum strength of 3,000 p.s.i. at 28 days. The average in Fig. 7 is close to 4,000 p.s.i.

The results of concreting were finally investigated by taking cores out of the finished structure. Examination of these cores showed that when the placement conditions are considered, the concrete must be described as being uniformly good beyond what might be expected. Absorption tests on the cores showed that the concrete had comparatively low absorption for the water-cement ratio and slump employed. Since the concrete is not waterproofed in any special manner, the low absorption results are of special interest.

Both surface inspection and compressive tests of cores showed what would normally be expected with average compressive strengths well over 4,000 p.s.i. Aggregate distribution was generally quite uniform and the amount of honeycomb disclosed was negligible.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the JOURNAL Supplement concluding this Proceedings volume

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of Two Papers by P. F. Girard and V. E. Gunlock, respectively:

Design of, and Construction of, Chicago's Initial System of Subways*

BY THOR GERMUNDSSON, R. B. YOUNG, M. O. WITHEY, J. A. McCARTHY, J. C. PEARSON, AND AUTHORS

CONVENTION DISCUSSION

Thor Germundsson†—I think it would be worth while to know a little more about the tests. Mr. Girard said the column loads and deflections were accurately determined by field measurements. In this subway there were a great number of different shapes of sections. Was any attempt made to check the measurements against the original assumption as to the distribution of moment in the concrete sections? It reads as though you had just put strain gages on the structural steel columns but I would like to know if anything else was done to check up on the elastic behavior or on the distribution of moments. In regard to Mr. Gunlock's paper—I understand that the hardest job was to fill the tops of the arches with concrete, that you went to considerable trouble, using either pneumatic or pumperete methods. Did you make any observations to show just how far shy you were in filling the sections? In other words, there might be a considerable void, maybe two inches below the steel lining. Was there any way of checking up on how much that was short of the goal; were you sure it was completely filled with grout, and was the grout checked in such a way that you were sure it was approximately of the strength of the concrete below it?

Mr. Girard—I can add a little to what I said about the loads on the columns. Most of the loading was around 700,000 lb. per column

*ACI JOURNAL, Feb. 1941; *Proceedings* V. 37—Mr. Girard on Design, p. 473, Mr. Gunlock on Construction p. 497. The convention discussion was consolidated following the presentation of the two papers.

†Portland Cement Assn., Chicago.

(700 tons per bay of 2 columns) and we did make measurements in the field to determine whether that 700,000 lb. was approximately correct.

Mr. Gunlock—One of the first things we did when we started construction of the subway was to inaugurate a soil testing section which made extensive tests not only on the consistency and bearing values of the soil, but also on the reactions or loads that we get from that soil. We left out the secondary concrete lining in two places for a section about 30 ft. long. That is we installed only the primary steel lining and have not installed the secondary concrete lining. We have one test section of the horseshoe type where we used the hand mining, and one section of the circular type where we used the shield mining. We have been taking strain gage readings in the steel lining members to determine the amount of moment and its direction, taking readings often enough to determine the change. We have also taken very careful measurements on diameters so that we can record changes every time they take place and find out what the movement is and what the pressure of the soil will be. In the case of the steel columns that we were discussing, we put strain gage points in these steel columns immediately after they were set, to get a zero reading. We added a load to these columns by the casting of the concrete arch. This load steadily built up over a short period while the subway was still under compressed air. Immediately after the compressed air was removed, we noticed a large increase in the compression of these columns—to 750,000 lb. per column; then it began to decrease. We attribute that to the faster movement of the vertical loads and the slower movement of the horizontal loads. As the horizontal loads came on the side, it lessened the amount of load from the top, so that they are now practically at rest with 700,000 lb. per column, which I believe Mr. Girard said was within ten per cent of the design load. We have made the same kind of test on the concrete lining; we have set copper inserts with accurate points scratched on them so that we can measure deflections in arches and changes in vertical and horizontal diameters. We are doing that on a number of sections so that we will know what effect soil pressure will have on the structure.

One of the most difficult things we encounter in placing the concrete in tunnels is the filling of the arch with concrete. We adopted pumping machines and pneumatic placing machines, and in both instances those machines pumped the concrete through a pipe up into the top of the arch. This works fairly well when filling the side walls; and it works well until we get the concrete near the top. Our shield sections, with a radius of nearly twelve and a half feet, give us a rather flat

arch to force that concrete under. If we make the concrete stiff enough so that we can blow it in with pressure to stand up on the slopes, then we have trouble out on the sides where the slope of the concrete will probably be 40 or 45 degrees. If we make it plastic enough so that it will flow into the sides, then we have trouble holding it in the arch, as it runs off with each slug of concrete we blow in there. The happy medium, of course, is to have it where it will just stay up on a slope in the arch and still be plastic enough to fill the sides. We approached that point with our mix using about six gallons of water per bag of cement, which gives us about a four-inch slump. This is hard to control; concrete with the four-inch slump does not act the same at all times.

The steel lining in the horse shoe shaped tunnel consists of a 6-in. arch rib. In the shield tunnels this rib section is 10 in., so that our problem is to force concrete up between these sections which are 10 in. deep and 33 in. apart; and to make it worse, we have an inside ring about ten inches wide on the inside of this rib section. We have to make the concrete flow up between these rib sections and stay there. We have tried a number of methods. With 10-ft. pipe sections there were sometimes voids in the top of the tunnel and we changed to 5-ft. sections and do not get voids. We formerly used very little air pressure with the pumpcrete machine, but now in placing the arch concrete, we boost the concrete out of the end of the pipe with air pressure. By putting an air hose connection near the bottom of the pipe where it curves up into the forms, the pipe is pumped full until concrete flows out the end, and then a pressure of 100 psi. is introduced into the pipe and that blows the concrete back into the arch. If we can make the concrete flow in with a slug and it does not go down afterwards, we can fill the arch completely. We have many cores which show the arches completely filled, even in the shield mined tunnel with the ten-inch deep sections, but we do not always fill the arch with concrete. We expect about a three- or four-inch void in the shield sections. We have had 10-in. voids in the shield sections, but those are very rare. To fill the voids in the arch we put grout pipes through the concrete masonry at 5-ft. intervals. This allows a grout pipe for every other ring of plates. The steel web is burned out, a hole about 6-in. in diameter, so that the grout will flow through this web. We pump cement grout through these pipes until the concrete arch is completely filled. To insure strength comparable to the designed strength of the concrete, we watch the water content of the grout the same as we do the water content of the concrete. Originally we did not do this; we specified a one to three grout and there was not very close control of the

water. We took cores immediately after we had poured a short section of the subway and grouted it and we found considerable variation in the compressive strength of the grout; sometimes as low as 1000 lb. and sometimes well over 5000 or 6000 psi. Now we use a mix which has a water limit of six and a half gallons per bag of cement. We cannot use a one to three mix because that mix will not flow through the grout holes; it plugs them. The contractors have adopted a grout of one part of cement to two parts of sand with six and a half gallons of water, and we find that this gives us well over the required strength. We have been employing this method in all of the shield tunnels, and after we have grouted them we drill 2-in. holes through the top of the arch to investigate the voids, and have found them negligible.

As to the relation of the strength of the cores to the strength of the concrete, our average core strength is over 4000 psi. We have no regular age for those cores, but we take them as soon as we can after the other operations have passed the point so that we know as soon as possible what our conditions are. We investigate those cores for a number of things. We take them primarily to find out the extent of our voids in the arch, but in taking them we also investigate for compressive strength, absorption, density, freezing and thawing, and run other tests to determine concrete quality in the finished structure.

In our concrete cylinders we get an average compressive strength of nearly 4000 psi. in 28 days. The concrete test cylinder is merely a test of the mix. It is made and cured under laboratory conditions, it is rodded many times; its density is usually greater than that in the finished structure. In comparing cores with cylinder strengths: available data led us to expect strength of concrete in place lower than that of test cylinders. Still our cores show that our average strength in place is greater than the strength of our cylinders at 28 days. This may be due to a number of reasons, first that our cores in most instances will be two months older than our cylinders. Another reason is that we have ideal curing conditions, warm and very humid—at times the humidity is 100 per cent with a heavy fog in the air. The temperatures run up to 85 and 90. With that humidity and heat, high early strength in concrete is to be expected. We have a concrete in the structure more dense than expected, slightly under 150 lb. per cu. ft. of concrete, which I believe is good for a structure of that kind. It is difficult to reach all the concrete in these arches; some is inaccessible, after the reinforcement, steel forms and concrete are placed. We can only vibrate it externally or design our concrete mix so that it will flow and hope for the best. That is the reason we have taken cores, to

find out what our actual conditions are, and although striving to improve them daily, are happy that they turned out as well as they have.

A Member—What is the size of the core?

Mr. Gunlock—Originally we took 6-in. cores; those were expensive and told us little more about the structure than the 4-in. cores that we are now taking. When we take a core, we go right on through, whether we encounter steel members or not, and we take these cores at what we think will give us the worst concrete in the structure. We have not investigated where we can see the concrete going in and know it is good; we have investigated doubtful places.

A Member—Can you give any reason for the wide variation in the slump and such a small variation in the water-cement ratio?

Mr. Gunlock—We do have a wider variation in the slump, than in the water-cement ratio, and sometimes the slump does not correspond to the water-cement ratio. I do not believe that a slump should be considered an important test of concrete. It is a useful test primarily in the first batch mixed. Our mixer inspector makes a slump test to compare his results with the results that he knows he should get from the data sheet furnished him by our proportioning engineer. If the slump shows that he is way off, for some reason, then he immediately contacts the proportioning engineer and another test is made to be sure of the data, and sometimes we can detect mechanical difficulties in the water control by the slump test. If we are running six gallons of total water in a batch and getting a seven inch slump, we know immediately that something is wrong and start checking with the mixer, for even with our present day machines we find that the water control gets off very often. It will not *stay* in adjustment, and we check it by weighing the water on platform scales periodically, but between those periods something may happen which the slump test detects. It also lets the mixer inspector know about what consistency of concrete he is to send into the tunnel until such time as he hears from the inspector on the heading who will tell him whether to add or subtract water to get the concrete he wants. That has not answered your question very well—why we have that difference. You may take a slump which will be four inches and I take a slump of six inches—our tests will never do away with that personal equation. Sometimes the batch with the known water content, does not show the slump we expect. Why that happens, we do not know, but we may run tests for a period of a week or two weeks without knowing what is causing it. We do hit periods in which the slumps of the concrete from all of the mixers will not react as we expected; in that case we start looking for some differences in the sand. Now one little change in the sand will

sometimes cause a variation in the slump. We have been able to control that to some extent by controlling the fines in our sand.

*F. E. Richart**—Mr. Gunlock mentioned the use of the air jet in placing concrete. I wonder if he will tell us more about it? Is there any evidence of air pockets due to its use.

Mr. Gunlock—We have used the air jet primarily to move concrete. We start with the concrete pipe in the top of the arch near the back of a section; the form may be set up for fifty feet; the concrete runs over the form and down on the side, and the slope is determined by the consistency of the concrete. Sometimes the concrete flows readily; at other times we have difficulty in getting it to flow. Use the air jet which consists of a straight piece of pipe, usually of about ¾-in. diameter, connected to a high pressure air hose with valve control. The jet is introduced into the concrete and a small amount of air is turned on; bubbles of air seem to enlarge as they come up through the concrete and as the pressure becomes less, the whole mass of concrete is shaken—a much greater mass than we are able to shake with the ordinary internal or external vibrators. In shaking that mass it also moves and occupies less space; because more dense. We then used the air jet extensively on inverts so we could watch its operation before we allow its use on the arches. We have taken samples of concrete with and without air jets and noticed no appreciable difference in air pockets in the concrete. We introduce air into our concrete when we place it with the mechanical pneumatic placer, and we also introduce air with the pumpcrete machine in our booster, and that air probably causes more air bubbles in concrete than the air jet. We do not allow the use of the air jet alone in any subway construction. When using it on the invert to move concrete, it moves it out on the side and gets it all around the steel legs. We want it to grip the legs securely and we use it to get the concrete to flow where we have dense reinforcement but we also use other types of vibration with it. It has worried us some that we might be introducing air into the concrete which would not get out, but tests have not shown this to be the case.

President Young—I might add that we have experimented a little with the air jet and find it very useful on occasion although we are cautious because we are not sure what the ultimate effect will be. It seems to bloat the concrete a little on occasion with some residual air.

M. O. Withey†—I would like to ask Mr. Gunlock in connection with the jacking up of these columns in the underpinning of a building if the entire load was carried into the column by the reinforcement

*University of Illinois, Urbana.
†University of Wisconsin, Madison.

alone. If so did you observe any distress in the column just above the jacks due to the imposition of the load?

Mr. Gunlock—In that case the installation I described was merely to take hold of the columns. Our jacking, of course, was done below this three-inch bedplate, and in jacking that we noticed no distress at all in the concrete at the column, we noticed no effect of the load we had picked up, but we did notice the effect on the brick work between the columns. We opened up some fine cracks there, which is not unusual in underpinning work. In nearly every underpinning job we have done or that any of the contractors have done, when we raise a building or hold a building whose foundation is sinking, we do introduce some cracks in the masonry or brick work, but we noticed no distress in the column above where we took hold. It was a twelve-story hotel building and the loads in the column were 550,000 lb. It was designed with a high carbon steel and a high compressive strength in the concrete and a very high compressive strength in the reinforcing.

Mr. Girard—This hotel is only 40 ft. wide and 12 stories high, with unusually large concrete columns.

Mr. Gunlock—We went directly under this building.

Member—I would like to ask Mr. Girard if the assumptions of design practice connected with the Chicago subway were much at variance with those used in the New York subway.

Mr. Girard—I thought probably some one might bring up that question. I don't know how many of you are familiar with an article recently written by Alfred Brahdy, designer for the Board of Transportation. He was kind enough to send me a copy of this article and as soon as I received it, I checked over our designs in comparison with those used in New York, and I find that in most cases we are using practically the same. The structural steel in tension is 20,000 lb. in New York while we are using only 18,000 lb. They are using 850 lb. compression on concrete where we are using 1200 lb., but throughout the rest of the stresses and most of the load assumptions, they are practically the same.

*J. A. McCarthy**—Mr. Gunlock has used the word "primary" in connection with steel lining and "secondary" in reference to concrete. I would like to know in what sense he uses these words. Apparently there was some sort of a break between the steel and the concrete. How is the load transferred from the outer steel section to the concrete?

Mr. Gunlock—The words "primary" and "secondary" are used in this case to indicate the chronological order in which they are installed. Primary lining means that it is the first lining installed, and secondary

*University of Notre Dame, Notre Dame, Ind.

means that it is the second. The first one is the steel and the second is reinforced concrete. In horse shoe sections in which we used the open face mining, we attempt to place the concrete lining as quickly as possible after the excavation is done, so that our steel lining will not support the loads for very long periods. This is because we put in as light a lining as we feel is safe. We depend on air pressure to hold in most of the places during the period and we do not wish to go to the expense of putting in a heavy steel lining because it is a temporary lining and is not figured as a part of the structure. Is that right, Mr. Girard?

Mr. Girard—The primary lining is designed for about 50 per cent of the actual load.

Mr. Gunlock—In the case of shield mining, in which we install a heavy primary lining, we do not worry about the time elapsing between the installation of the primary lining and the secondary lining. We insist there that the contractor keep his concrete at least 100 ft. behind the shield operations.

*J. C. Pearson**—I would like to say something in reference to the variations in slumps appearing in Mr. Gunlock's records. It seems that these variations are evidence of the honesty of the inspectors in setting down what they observed instead of what they expected. Anyone who has made slump tests conscientiously must realize that the probable error of a single test is large. Very frequently, except possibly in wet or very plastic mixtures, the top of the cone falls off to one side—not necessarily a complete collapse, but quite out of symmetry. When the slump occurs in this manner, leaving no horizontal surface, and frequently leaving an irregular surface with a considerable pitch across the axis of the cone, the measurement becomes arbitrary. The point is that we have no really adequate measure for the working consistency of the concrete, and this is particularly true in the laboratory where some such device has to be used for consistency control in the fabrication of test specimens. The problem is unsolved when we are dealing with stiff, vibrated concrete.

President Young—I wonder what Mr. Pearson would think of a man who recorded a slump of $\frac{1}{8}$-in?

Mr. Gunlock—I hope you did not get the impression that I do not value slump tests. Taking of a slump test is important in that it will tell you something, but we should never try to determine our water-cement ratio by taking a slump test. If we want to know the water-cement ratio we should determine that by more accurate means.

*Director of Research, Lehigh Portland Cement Co., Allentown, Pa.

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Design of Light-Weight Zonolite Concrete Mixes*

GREGORY P. TSCHEBOTAREFF†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The results of a relatively limited number of tests with light-weight Zonolite concrete mixes are given to illustrate peculiarities of such mixes which necessitate a special approach to the problem of their design. The investigation did not cover all of the factors which are likely to affect the properties of such light-weight concretes, but some of these are listed and briefly discussed.

Zonolite is a variety of vermiculite obtained by sudden heating of the ore. Some of the water contained between the mica flakes is thereby transformed into vapor and expands the flakes, giving the individual grains of the aggregate an accordion-like, very porous and compressible structure. Light-weight concretes made with aggregates of this kind have excellent thermic insulating properties and can be used for many practical purposes. In March, 1939, the writer completed a brief investigation[1] of design methods of light-weight Zonolite concrete.

The unit weight of these aggregates in a loose state can vary considerably, depending on their grading and on the type of ore originally used. If sieved aggregate of sizes smaller than ¼ in. but larger than 1/16 in. is used, the loosely measured material will weigh about six lb. per cu. ft. The Zonolite tested was of smaller sizes and of better grading. Its grain size distribution curve is shown in Fig. 1. Loosely poured in it weighed 15 lb. per cu. ft., and shaken down and very lightly tamped—18 lb. per cu. ft. More vigorous tamping could decrease its volume further. For this reason the mixes were propor-

*Received by the Institute Sept. 3, 1940.
†Assistant Professor of Civil Engineering, Princeton University.
[1]For Munn & Steele, Inc., 130 Lister Avenue, Newark, N. J.

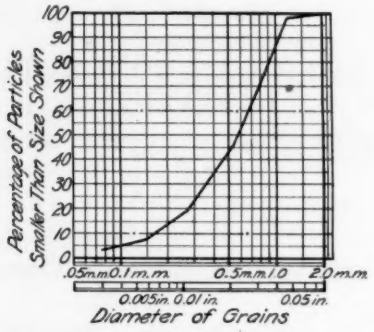

FIG. 1—GRAIN SIZE DISTRIBUTION CURVE OF THE ZONOLITE TESTED

tioned by weight and not by volume, although the latter method of proportioning is still currently given in most commercial pamphlets dealing with similar aggregates.

The very low unit weight of this aggregate prevented the effective use of vibration for the compaction of the concrete mix, especially at low cement contents and without excessive addition of water. The mix simply would not shake down, visibly retaining the original honey-combed structure of the loose aggregate. Considerable quanties of water had to be added to permit noticeable compaction by vibration. Since it was obvious that such excess of water would considerably weaken the cement paste and therefore the bearing matrix of the concrete, vibration was not given further consideration as a method of molding. Recently reported conclusions[2] concerning the advantages of vibration for light-weight aggregates, all of which weighed over 45 lb. per cu. ft. in a damp loose state, are not applicable to the three to ten times lighter "feather-weight" aggregates of the vermiculite type.

The usual procedure of concrete mix design,—that is the indication of a definite water/cement ratio and its correction for water carried on the surface or absorbed by the aggregates, could not be used in this case. The main difficulty here lay in the determination of the amount of water absorbed by the Zonolite. In the case of sand, gravel, slag, pumice or other solid aggregates this amount of water of absorption has a fairly constant value depending only on the porosity of the aggregate substance and on its state of dryness. In the case of Zonolite or other vermiculite aggregates the amount of water absorbed could be strongly affected by another variable,—that is the pressure

[2] "Tests on Concrete Masonry Units Using Tamping and Vibration Molding Methods," by K. F. Wendt and P. M. Woodworth, JOURNAL American Concrete Institute, Nov. 1939, *Proceedings* Vol. 36, p. 121.

applied to the Zonolite while compacting it, since this pressure squeezed out some of the water absorbed in a loose state.

For this reason none of the standard tests for the determination of the water absorbed by aggregates could be used for any accurate determinations. They, however, gave a rough indication that the amount of water absorbed in a loose state was of the order of dimension of 100 per cent expressed in terms of the weight of the aggregate.

The compaction of the Zonolite itself appeared to be greatly facilitated by a certain addition of water. In this respect the behavior of the Zonolite was totally different from that of the usual concrete aggregates (that is, from the "bulking" of sands) and to some extent resembled the behavior of clay or loam soils for which there exist definite so-called "optimum moisture contents" at which a maximum density, and therefore strength, can be obtained with the same method of compaction[3], [4]. The compaction of the Zonolite naturally lessened the volume of voids it contained and thereby increased the unit weight of the concrete, which nevertheless remained extremely low; at the same time, it increased the strength of the concrete.

The investigation described in this paper was intended to cover only the determination of the advisable water content for mixes of different proportions of cement and aggregate using the same method of compaction for all mixes. The tensile strength was taken as main criterion for this determination, chiefly because of the greater ease and accuracy of briquette testing under field conditions, which this investigation was intended to duplicate. Six briquettes were prepared for each mixture, of which three were tested at seven days and three at 28 days. A few slight inaccuracies in the results of the tests probably are due to the comparatively small number of the briquettes tested. They nevertheless gave quite a clear picture of the essential points. Tests of 18 2-in. cubes were made as a check to get an approximate idea of the compressive strength to be expected. The mixing and molding was done in accordance with the A.S.T.M. specification C77-39, except that the pressure applied by both thumbs was greater, varying from 50 to 70 lb., than the one specified, 15 to 20 lb. for ordinary cement or mortar briquettes. Further, the briquettes and cubes were kept in their molds for 24 hours in the moist curing room. They were then withdrawn from the molds and were kept for another 24 hours in the moist curing room. After that they were stored until tested in the normal air of the laboratory.

[3] "Fundamental Principles of Soil Compaction," by R. R. Proctor. *Engineering News Record*, 1933, Vol. III, pp. 245-248; 286-289; 348-351; 372-376.
[4] "Essentials of Soil Compaction," by C. A. Hogentogler, Jr., *Proceedings*, Highway Research Board, 1936, Vol. 16, pp. 309-316.

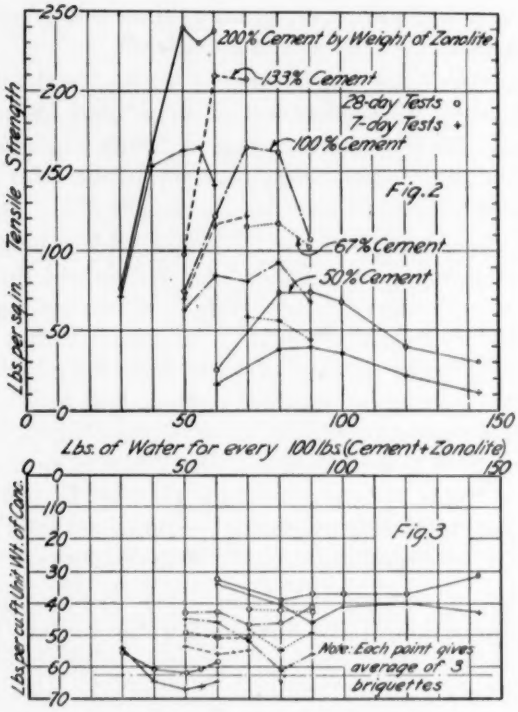

FIG. 2—EFFECT ON THE TENSILE STRENGTH OF THE AMOUNT OF WATER USED FOR DIFFERENT PROPORTIONS OF CEMENT AND OF AGGREGATE

FIG. 3—EFFECT ON THE UNIT WEIGHT OF READY CONCRETE OF THE AMOUNT OF WATER USED FOR DIFFERENT PROPORTIONS OF CEMENT AND OF AGGREGATE

The results of the tests are shown on Fig. 2, 3, 4, 5, 6, 7 and 8. It may be seen from Fig. 2 and 3 that in each mix the greatest tensile strength corresponds to the greatest unit weight of the mixed concrete. Fig. 4 and 5 show the manner in which both the maximum tensile strength and maximum unit weight of each mix increase with the increasing cement content of the mix. Fig. 6 shows the rapid decrease in the quantity of water producing the maximum tensile strength of each mix with the increasing cement content of the mix. Fig. 7 shows the quantities of cement and of Zonolite aggregate per cubic foot of mixed concrete of greatest density in different mixes. It may be seen that the quantity of aggregate decreases very slightly with increasing cement content. Greater quantities of cement paste, therefore, serve mainly to provide a better coating to the individual scales of the

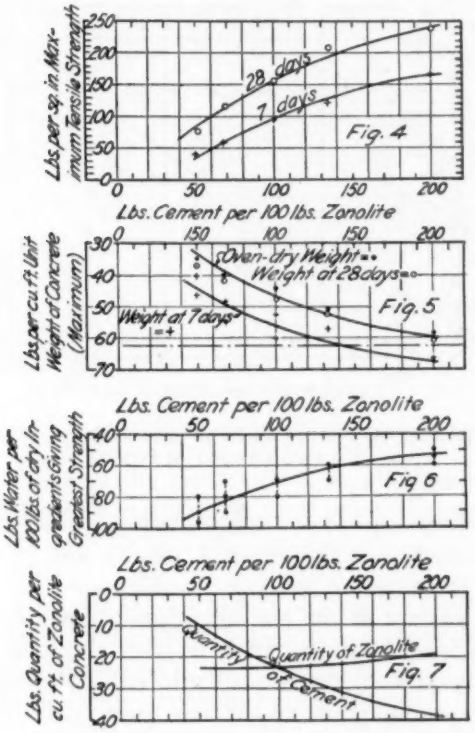

FIG. 4—MAXIMUM TENSILE STRENGTH OBTAINED FOR DIFFERENT MIXTURES OF CEMENT AND OF ZONOLITE

FIG. 5—MAXIMUM UNIT WEIGHTS OF MIXED CONCRETE MADE WITH DIFFERENT PROPORTIONS OF CEMENT AND OF ZONOLITE

FIG. 6—QUANTITY OF WATER FOR DIFFERENT MIXTURES OF CEMENT AND OF ZONOLITE WHICH PRODUCED THE MAXIMUM TENSILE STRENGTHS SHOWN ON FIG. 4

FIG. 7—QUANTITIES OF CEMENT AND OF ZONOLITE PER CUBIC FOOT OF READY CONCRETE OF GREATEST DENSITY IN DIFFERENT MIXES

accordion-like grains of the aggregate. Fig. 8 shows the maximum compressive strengths obtained. Tht ratio of the tensile strength to the compressive strength was found to be fairly constant, varying from 1:3.2 to 1:3.7.

In addition to the variables considered in this investigation, the following factors are likely to be of considerable importance in the design of light-weight concrete mixes of similar type:

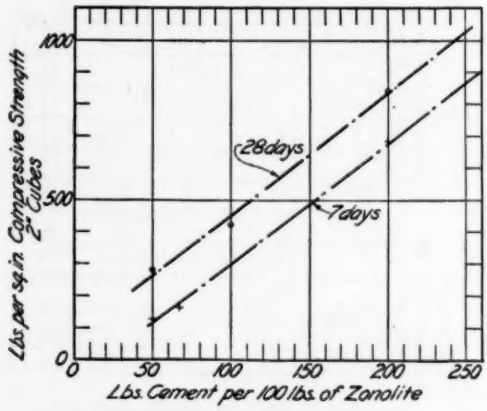

FIG. 8—MAXIMUM COMPRESSIVE STRENGTH (2 IN. CUBES) OF DIFFERENT CEMENT-ZONOLITE MIXES

1—The pressure applied for compaction purposes during molding. The writer noticed that the greatest density and strength of a mix was obtained for a water content at which moisture just began to appear on the surface of the molded briquette at the end of its compaction. This would indicate that, as with clay and loam soils, the greatest density and therefore strength was obtained when all the voids left after compaction of the mix were just filled with water. Excess water prevented efficient compaction. Since the amount of these remaining voids depends, among other things, on the pressure applied during molding, it follows that the quantity of water required to give maximum strength to a definite mix must also be a function of the pressure applied during molding.

2—The grading of the aggregate may also be of importance.

3—The origin of the ore is of importance, since different ores are apt to expand differently during production of the aggregate.

4—Moist curing of such light-weight concrete probably is of lesser importance than for ordinary concretes, and that becasue of the large amount of voids filled with water.

5—The use to which the lightweight concrete will be put and the conditions of placing the mix on the job are important considerations. In some cases, as when thermic insulation of large horizontal surfaces is the main requirement, it may prove advisable not to use prefabricated units, but to place the mix in-situ. Tamping may then become relatively expensive and fluid mixes are sometimes resorted to as an alternative method of placing, since strength is of secondary impor-

tance. However, it may be that even the minimum strength required under such conditions will be obtained only through a large increase of the quantity of cement in the mix. For instance, a sample of similar lightweight concrete which has been placed on an actual job in a semi-fluid state was claimed to have approximately the same strength as the tamped test-specimens of corresponding unit weight (and therefore porosity) designed by the writer and described in this paper (350 p.s.i. at 28 days and a unit weight of 36 lb. per cu. ft.) But the proportion of cement was increased fourfold for the fluid mix. Thus decreased labor cost as compared with tamped concrete was at least partially offset by the much smaller yield. On jobs where machine-molding of units is possible, the method of design as described may prove of advantage.

CONCLUSIONS

1—The great variety in the possible density, grading and grain structure of Zonolite or vermiculite light-weight aggregates makes it advisable to determine by trial the quantity of water to be added to a mix on each job.

2—Vibration does not appear to be an advisable method of molding of concrete masonry units with mixes using such extremely light, "feather-weight" aggregates.

3—The pressure applied for compaction purposes during molding, and the grading of the aggregates are among the factors affecting the choice of the quantity of water to be used. The most advisable grading and amount of pressure to be applied are yet to be determined and should be studied in respect of aggregates produced from extreme types of ores used for the purpose.

4—The design methods described are most likely to find application where relatively greater strengths are required and where machine-tamped, prefabricated masonry units are advantageously employed for partition and interior walls.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by June 1, for publication in the* JOURNAL *Supplement concluding this Proceedings volume

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information. No formalities; just write a letter to "the Editor."

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Effects of Materials on Cracking Tendency in Dams? (37-67)

In the hope of prompting discussion, from competent observers, of the cracking tendencies in dams, in relation to materials used, I submit these notes on the now completed Conchas Dam.

At the confluence of the Conchas and South Canadian Rivers 30 miles northwest of Tucumcari, New Mexico, it was designed and built under the direction of the Corps of Engineers, U. S. Army, to supply flood control and irrigation benefits.

The project consisted of a gravity concrete main dam, earth wing dams, earth dykes, earth saddle dam, concrete emergency spillway, and irrigation head works, involving 1,100,000 cu. yds. of excavation

and placement of 836,000 cu. yds. of concrete, 2,815,000 cu. yds. of earth fill, and 786,000 cu. yds. of rock fill.

The main dam holds possibilities of interest in the concrete field because of its low cement content and, it appears, a decreased tendency toward cracking.

The main dam contains 755,000 cu. yds. of concrete, maximum height 235 ft., crest length 1,250 ft., and maximum thickness or cross section of the dam at base 197 ft. Modified cement was used entirely —785,000 bbls.

Construction data and the concrete data in Table 1 were supplied by the Army Engineers.

TABLE 1—AVERAGE CONCRETE TEST DATA

	Interior of Dam	Interior of Dam	Face of Dam
Bbl. cement per cu. yd.	0.75	0.9	1.0
Cu. yd. of concrete	340,000	195,000	220,000
W/C gal. per bag	7.80	7.00	6.50
Average slump, inches	1–1½	1–1½	1½–2
Mix by weight	1:3.46:9.35	1:2.62:7.81	1:2.49:6.78
Effective maximum size of aggregate	4"	4"	4"
Compressive strength (28 day) p.s.i.	2750	3465	4110
Tensile strength (28 day), p.s.i.	240	269	275
Shearing strength (28 day), p.s.i.	1075	1294	....
Flexural strength (28 day), p.s.i.	522	556	
Unit weight, lb.	148.99	149.42	149.55
Modulus of Elasticity (28 days)	....	4,000,000	....
Poissons Ratio (28 days)	....	0.22	....
Thermal Coefficient of expansion (7 mo.)	....	.0000052	....
Method of Compaction—Internal vibration.			

TABLE 2—PHYSICAL AND CHEMICAL PROPERTIES, AND SPECIFICATION REQUIREMENTS FOR MODIFIED CEMENT

Chemical Analysis	Average of Acceptance Tests	Specification Requirements
Ig. Loss	0.86%	4.00% (max.)
Insol. Res.	0.11%	0.85% (max.)
SO_3	1.69%	2.00% (max.)
MgO	1.77%	5.00% (max.)
Compound Composition		
C_3S	44.50%	55.00% (max.)
C_3A	5.90%	8.00% (max.)
Physical		
Soundness	O.K.	A.S.T.M. Boil
Initial set	about 2:30—4:30	45 min. (min.)
Final set	about 4:30—6:30	10 hrs. (max.)
Specific surface	1899	1700—2200
*Compressive strength**		
3 Days	1575	750 (min.)
7 Days	2352	1500 (min.)
28 Days	4276	2500 (min.)

*Compressive strength tests are by A.S.T.M.-C109-34T method.

DATA ON CONCRETE LIFTS

In general all concrete was placed in five foot courses or lifts. A minimum time of three days was specified between lifts, however,

construction operations were such that the usual time between lifts was approximately five days. Monolith widths varied from 40 to 50 feet except in special instances. All monoliths with the exception of those at the abutments were cast from heel to toe without the use of construction joints. At the foundation this distance approximated 200 feet. Air temperature conditions were:

Air Temperature Data	1937	1938
Mean Annual Air	58°F.	58°F.
Minimum Air	2°F.	4°F.
Maximum Air	103°F.	104°F.

The record of adiabatic temperature rise of mass concrete in degrees F. is tabulated:

Cement Content—BBl. per cu. yd.		0.75	0.9
1 Day	after placing	21.7	30.6
2 Days		32.0	43.0
3 Days		37.6	48.5
7 Days		42.2	52.0
14 Days		42.5	52.2

DATA ON CRACKING

A crack across a monolith parallel to the axis of the dam, extending from the foundation, where it had a width of approximately $\frac{1}{8}$ in., upward about 55 ft. where it ran out, was grouted.

Minor horizontal cracks (rarely more than $\frac{1}{32}$ in. wide and slight penetration) developed along lift joints on the downstream face of the dam. Cores were drilled to intercept some of the widest of these cracks at a depth of about 5 ft. from the face and no trace of them were found.

Crack surveys were made in a few of the galleries. In general it was found that the amount of checking or surface cracking noted in the galleries was closely related to the time of form removal, cold air, drafts, etc., suggesting that construction procedure has an important bearing on cracking.

Artificial cooling of concrete was employed to a limited extent in the two abutment monoliths only. These monoliths were not placed continuously from heel to toe but were cut up into a number of sections by means of construction joints. To make these sections act monolithically under load it was necessary to grout the construction joints after the concrete had taken its temperature shrinkage. As an aid in hastening this condition a series of cooling pipes were embedded in certain sections of both monoliths. These cooling coils consisted

of ¾ in pipe placed in horizontal runs spaced 5 feet vertically and 5½ feet horizontally. Cooling was accomplished during winter months by pumping river water through the coils immediately after placing the concrete. This appeared to be very effective in extracting the heat from the concrete.

In spite of the reduction in the cement content of Conchas Dam concrete the wet concrete was plastic and workable without excessive segregation or bleeding. The hardened concrete is impermeable and the strengths satisfactory. At the same time the reduction in cement content effected a considerable economy in construction cost.

The two most favorable appearing competitors of modified cement for mass concrete are low heat and portland-pozzolan cement but from a manufacturing view point drawbacks to the latter are the additional steps involved in obtaining, or preparing, and incorporating the pozzolan in the cement. An objectionable manufacturing feature of low heat cement is the toughness of the clinker which requires a great deal of power and grinding time.

During the entire period of construction Captain Hans Kramer was district Engineer in charge of construction and Ralph Johnson was concrete technician at Conchas Dam.—S. L. MEYERS*

*Chief Chemist, Southwestern Portland Cement Co., El Paso, Tex.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY, 1941*

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Expansion of concrete through reaction between cement and aggregate

THOS. E. STANTON, *Proc. Am. Soc. C. E.* Vol. 66, No. 10, Dec. 1940, pp. 1781-1811.
Reviewed by H. J. GILKEY

Recital of tests which appear to have demonstrated that disruptive action in concrete may result from reactions between cements of relatively high alkali content and aggregates containing certain types of shales, cherts and impure limestones. The test procedure evolved consists of curing specimens of concrete in sealed containers at normal temperatures. Alternately wet and dry, continuously immersed or continuously dry were not effective for exposing adverse symptoms within reasonably short periods. Moist storage in an unchanging environment was satisfactory. The difficulties have appeared throughout coastal regions of California in connection with some important structures. The high alkali cements proved unsatisfactory only with certain aggregates and the objectionable aggregates were satisfactory with low alkali cements. Amounts of alkali (Na_2O or K_2O) below about 0.5 per cent appear not to be harmful. This cause of major concrete deterioration is of recent discovery and constitutes a finding of the utmost importance.

Filling and sealing of expansion joints (committee report)

JOHN S. CRANDELL, Chairman, University of Illinois—at 20th Annual Meeting Highway Research Board, Dec. 1940. HIGHWAY RESEARCH ABSTRACTS

This report describes several experimental joint projects in Illinois which are being observed by the University of Illinois and the State Highway Department. A brief review is given of the current practice as to the type of joints and joint compounds in use during 1939. The following types of joint fillers are discussed: (a) tar; (b) tar plastic; (c) poured rubber; (d) various asphaltic fillers; (e) premolded rubber; (f) boards; (g) oil-latex; (h) chlorinated tar and rubber; and (i) synthetic rubber and asphalt. The effect of the surface texture of the joint due to finishing on the adhesion of the filler is commented on and a type of the joint cutting device to overcome this fault is described. The committee comments that a stage in our transport has been reached which is different than that of 10 years ago and the tendency of fillers to be whipped out of joints by high speed trucks and cars with

their intricate tire tread design, is much greater today. A sealing material has to withstand not only this whipping action but the weather, the effects of freezing and thawing, the contraction and expansion of the concrete slabs and other items which tend to break down the filler. The ideal filler is yet to be found and the ideal method of cutting joints or preparing them for the filler is also yet to be developed.

Reinforced portland cement concrete in cold weather

RUSSELL V. ALLIN AND H. H. TURNER, *Cement and Lime Manufacture*, Vol. 13, No. 11, pp. 198-203, Nov. 1940. Reviewed by J. C. PEARSON

From a half-dozen references, the authors summarize fairly well established facts regarding the effects of low temperatures, freezing, necessity for protection, etc. of freshly placed concrete. Tests were made (a) on cubes wholly or partially frozen and tested after various periods of curing; (b) on cubes made at various temperatures by preheating water only and subsequently exposed to conditions similar to those in frosty weather. Results: Concrete frozen before setting will harden to a large extent but suffers permanent loss of strength up to 50 per cent, and increased porosity; concretes placed and exposed at temperatures between 50° F. and freezing hardened normally if freezing did not occur. Concrete mixed and placed at 45° and left in air at 26° reached the freezing point in 8 hours and ultimately showed a loss of 30 per cent in strength. These tests were confirmed by job observations, and the general conclusion was reached that concrete must be completely protected from freezing during construction. For the English climate, preheating of water only is considered adequate for satisfactory placing temperatures, which should not be high—50° to 65° being recommended for concrete to be placed between 40° and 30°. Complete protection by coverings for at least two days is recommended, assuming that minimum temperatures are not more than 10° below freezing.

Freezing and thawing investigations

F. V. REAGEL, Engineer of Materials, Missouri State Highway Department—at 20th Annual Meeting Highway Research Board, Dec. 1940. HIGHWAY RESEARCH ABSTRACTS

The scope and purpose of this investigation is covered in the early part of the report by a statement of the following questions: (1) What weathering conditions are the major factors causing deterioration of concrete? (2) To what cycles of freezing and thawing may concrete be subjected in service? (3) If the freezing and thawing test is to be used as a measure of durability, what cycle of freezing and thawing should be used? (4) What are the effects of various curing treatments prior to freezing? (5) How shall the disintegration of concrete in a freezing and thawing test be measured? (6) Can the effect of aggregates on the durability of concrete be predicted by a freezing and thawing test?

The report then offers data obtained in connection with research directed toward producing a yardstick by which some accelerated measure of concrete durability may be obtained. The data are based on tests made on concretes fabricated from a fixed mortar of good quality and coarse aggregates of varying quality. The freezing and thawing cycle used is of a fixed length composed of 10 hrs. freezing in air and 2 hrs. freezing in water, using varying thawing and freezing temperatures. These data are discussed briefly to point out some of the interesting trends and implications and with no attempt to draw any final conclusions, as the paper covers current and continuing but not completed research.

Investigational concrete pavement in Kentucky

F. P. Anderson, 2nd, Kentucky Highway Department—at 20th Annual Meeting Highway Research Board, Dec. 1940. Highway Research Abstracts

This experimental joint spacing project consists of seven different combinations of spacing of expansion and contraction joints with and without load transfer bars in the contraction joints. This is cooperative project with the Public Roads Administration and meets their experimental program requirements as well as the general alignment and topography requisites. The project is in Daviess County, approximately six miles from Owensboro on State Route No. 71. The subgrades consisting mainly of silty loams with some clay loams and classified as A-4 and A 4-6 were compacted with a sheeps-foot roller at optimum moisture content in 12-in. layers. The volume change of the subgrade is comparatively low, varying from 7 to 12 per cent.

The aggregates used were Ohio River sand and gravel and the Kosmos brand of cement was of the moderate heat of hydration classification. The concrete was cured with one layer of wet burlap overnight and then three days of Sisal Kraft paper. The concreting started July 8 and was completed August 16 for the 6.3 mi. of 20-ft. pavement. The average slump of the concrete was 2 to 2½ in. at time of placing and the 60-day cores averaged 4800 p.s.i.

The report contains a table showing the average opening of the joints between the initial readings in September and those taken in November. Another table shows the change in joint openings from morning to afternoon on a late November day. The report concludes that the project is too new to permit any conclusions to be drawn; however, it is felt that the 20-ft. joint spacing tends to decrease riding qualities and that the joints poured in August and September have opened sufficiently to permit surface moisture to reach the subgrade.

Experimental control of warping of concrete pavements

R. B. Wills, State Highway Engineer, Kansas State Highway Commission—at 20th Annual Meeting Highway Research Board, December 1940. Highway Research Abstracts

In an effort to substantiate the causes of the warping of concrete pavements and to develop practical and economical means of avoiding its occurrence on new construction, the Kansas Highway Commission built an experimental paving project, 5 mi. long, on Highway No. 10, east of the city limits of Lawrence, Kansas. Built in 1935 and divided into 20 sections, each of 13 of the 20 sections had some special type of subgrade treatment. Expansion joints were placed at intervals of 100 ft. and intermediate contraction joints were used in some sections. The subgrade treatments consisted of the following: (1) Drying soil to an extremely low moisture content for depths of 6, 12 and 24 in.; (2) Placing soil at plastic limits for depths of 6, 12 and 24 in.; (3) Ponding subgrade until soil moisture exceeds the plastic limit; (4) Use of granular blankets over wet and dry soil; (5) Use of bituminous membrane over dry soil; (6) Sprinkling dry soil continuously for 48 hours.

The observations made on the project have resulted in the following conclusions: (1) Warping of pavement slabs is produced by volume change of expansive soils caused by a differential in the moisture content brought about by surface water entering the subgrade through joints or cracks; (2) Warping of pavement slabs was avoided by placing the subgrade soil at a moisture and density such that no subsequent differential moisture content resulted from the entrance of water through joints or cracks; (3) Warping did not occur on expansive soils when the subgrade moisture was equal to or greater than the plastic limit; (4) A blanket course of

nonexpansive subsoil having moisture contents about ten per cent below the plastic limit; and (5) Warping caused by expansion of soil due to differential moisture contents was reduced in time as the moisture in the subgrade became evenly distributed.

Experiments with continuous reinforcement in concrete pavements

H. D. CASHELL, Public Roads Administration, and S. W. BENHAM, Indiana Highway Commission—at 20th Annual Meeting Highway Research Board, Dec. 1940. HIGHWAY RESEARCH ABSTRACTS

Last year there was described before the Highway Research Board, cooperative experiments with continuously reinforced concrete pavement sections laid near Stilesville, Ind. The present report contains the data obtained during the first two years of observation of these experimental sections.

Changes in pavement elevation have been generally small and there is nothing to indicate that these changes have affected the structural condition of the various sections.

The annual cycle of length change of the various sections shows that those of approximately 150-ft. length move with as much freedom as the very short sections. The movement of sections greater than 150 ft. long is apparently restrained by the subgrade and this restraint is progressively greater as the section length is increased.

In the long, heavily-reinforced sections many fine cracks have developed in the central area. In the sections of intermediate length containing ½-in. steel bars a moderate amount of cracking has developed, while a very limited amount of cracking has occurred in the short sections containing welded wire fabric. While the width of the cracks is slightly greater in the sections containing the smaller amounts of longitudinal steel there is no evidence of spalling, raveling, disintegration, steel failure or other structural weakness at any of the cracks. A relation between the length of the section as constructed and the average slab length (or a distance between transverse cracks) appears to exist. So far, no relation has been found between the average slab length and either the type or the amount of longitudinal steel.

Relative roughness determinations over the various sections show that the surface of the sections was very smooth after about 18 months of service.

Research on concrete disintegration

H. S. MATTIMORE, Pennsylvania Highway Department—at 20th Annual Meeting Highway Research Board, Dec. 1940. HIGHWAY RESEARCH ABSTRACTS

This is the report on the condition of some experimental walls built in a section of Pennsylvania where considerable difficulty had been experienced with the disintegration of concrete both in pavements and structures.

Forty-five walls were built under laboratory supervision. These walls were 36 in. high, 12 in. wide and 36 in. long and were so placed that 2 ft. of the height was in the ground, leaving a structure 1 ft. high subject to weathering conditions. The sections where these structures were placed had saturated soil most of the season and wet throughout the year. Recording thermometers placed at this site for five years, indicated extreme changes in temperature—as much as 70 degrees in 12 hours.

The report was mainly in the form of an illustration by slides of the condition of these walls after 10 yrs. exposure. This condition varies from some of the structures in an excellent condition to others which were disintegrated to a point where they were an absolute failure structurally.

Materials used in the construction were two different fine aggregates, one which tests indicated was an excellent sand for use in concrete while the other showed over 17 per cent loss by sodium sulphate test, indicating a non-sound aggregate. The coarse aggregate used was a crushed limestone and was the same in all walls. Cements of two different types were used, produced at different locations from different raw materials. These two cements show very little difference in composition, the main difference being in the alkali content.

Generally the structures in which Cement B was used have given good service while the structures in which Cement A was used varied considerably on structural rating at this 10-yr. period. The concrete mixed with Cement A and the low-loss sulphate sand was of good quality but this same Cement A when used with the high-loss sulphate sand produced concrete of poor quality. In case of Cement B the resulting concrete was good even with the high sulphate loss sand.

Investigational concrete pavement in Michigan

J. W. Kushing, Research Engineer, Michigan State Highway Department—at 20th Annual Meeting Highway Research Board, Dec. 1940. Highway Research Abstracts

Conclusions obtained by theoretical formulae on concrete pavement design according to many contemporary authors are not completely substantiated by practical observations in certain localities or under varying conditions. If it be true that there is a disparity between these conclusions and practical observations of certain localities—then before any locality can accept such conclusions it is necessary to determine the causes for such differences and where necessary, determine the proper conclusions for the given conditions. This status and the desire to improve practices impelled the Michigan State Highway Department to construct an experimental concrete pavement which would embody certain modern theories of design and methods of construction.

The road was constructed during the past year on M-115 in the north central part of Michigan's lower peninsula and is approximately 10.7 miles long.

This report discusses the planning and construction of the experimental road under three headings: (1) Features of Design; (2) Methods of Measurement; and (3) Incidental Studies. Under Features of Design are included such factors as (a) joint spacing; (b) expansion joints; (c) contraction joints; (d) cross-section; and (e) steel reinforcement. Methods of Measurement include: (a) slab movement; (b) joint width changes; (c) temperature of concrete; (d) moisture content of concrete; (e) strains in concrete; and (f) subgrade bearing capacity. Incidental Studies comprise a study of various construction methods, including: (a) stress curing of concrete; (b) mechanical spreading of concrete; and (c) the use of various joint sealers.

The report contains a table in which are listed all of the design features, also detail drawings of various joints and 31 pictures.

The facts and relationships obtained from this research project will undoubtedly assist in the classification of many controversial issues and will serve to aid various highway organizations in the development and improvement of concrete pavements.

Concrete mixing and placing on large dams

ADOLPH J. ACKERMAN, *Civil Engineering*, Vol. 11, No. 1, Jan. 1941, pp. 19 to 22 incl.
Reviewed by J. R. SHANK

A study of the construction costs of many of the large concrete dams recently built is presented. The author discusses the handling methods, organizations, and other features for each dam, and their effects on the cost. No general conclusions on costs were made. The author does, however, attempt to use a "Unit Cost Index" which is practically the cost of concrete per cubic yard in place. He divides these dams into two major classes: (1) Wide-river dams and (2) Canyon-type dams. In the first are included Conowingo, Safe Harbor, and Wheeler dams. In the second are Norris, Owyhee, Boulder, Grand Coulee, Tygart, and Morris dams. Adjustments were made to take care of the different wage scales and aggregate costs so as to get more accurate comparisons. The adjusted unit cost indexes for the first group were 25.43, 23.83, and 23.17 dollars respectively. For the second group they were 5.97, 6.47, 8.15, 7.01, 4.63, and 4.88 dollars respectively. Costs cannot be compared across these two groups because the inclusions are different for the two. In the first group a greater cost coverage was made. The costs of all labor and materials, all permanent equipment including spillway equipment, all temporary construction plant and equipment, camps, engineering supervision, insurance, and compensation were included. In the cases of the second group only those prices represented in the public bid schedules were used. These contractors' costs included labor, construction equipment, supplies, river handling, camp, supervision, aggregate production, overhead, and profit; but omitted the first cost of all permanent equipment, cement, and machinery. The figure 4.63 dollars for Tygart Dam was upped considerably by increased quantities during the execution of the job to a point which was more in line with the others. The low figure for Morris Dam was confirmed during the execution.

An important feature of this paper was a graph showing rates of placement of concrete at different stages of the construction. Graphs for a great many more dams were included on this chart than those listed for the two groups. This was also true for the discussion of costs and conditions for each dam which after all is the meat of this paper, and is not information that may be condensed appreciably for an abstract.

History and scope of cooperative studies of joint spacing in concrete pavements

E. F. KELLEY, Public Roads Administration—at 20th Annual Meeting Highway Research Board, Dec. 1940. HIGHWAY RESEARCH ABSTRACTS

The experience of the past has demonstrated that transverse joints must be used in concrete pavements if transverse cracks are to be avoided. Both experience and research have also shown that if transverse cracks are to be eliminated, the transverse joints must be so spaced as to divide the pavement into relatively short slabs.

Transverse joints in pavements are of two types. The first is the so-called contraction joint whose primary function is to reduce temperature-warping stresses. These stresses are much higher in long slabs than in short ones. The second is the expansion joint whose function is to provide space for the unrestrained longitudinal movement of the pavement with changes in temperature.

There can be no argument concerning the need for closely spaced contraction oints if the formation of transverse cracks is to be prevented but there is no agree-

ment among highway engineers regarding the need for expansion joints or their proper spacing when they are used.

For some years the Public Roads Administration has placed a limitation on the maximum spacing of expansion joints, the present limit being 120 ft., and, with some exceptions, has required the use of load-transfer devices in contraction joints in thickened-edge pavements. This means that dowel bars are required in transverse dummy joints.

These requirements have been questioned by some of the State highway departments and by the Portland Cement Association, with the result that early in 1940 the Public Roads Administration decided to sponsor rather comprehensive cooperative studies of the questions at issue. Accordingly, it proposed an investigational program involving the construction of experimental concrete pavements and offered to participate in their construction on a limited number of selected Federal-aid projects.

The report contains a description of the seven different sections included in this experimental program. Six State highway departments are taking part in the cooperative investigation. In addition to these tests the Public Roads Administration is carrying on a study of transverse dummy joints at their experimental laboratory in Washington. This special study is being made on six test slabs, 20 ft. wide and 30 ft. long and 8 in. thick (uniform). The variables included in the construction of these sections were such that they afford a means of studying the effect of the following factors on weakened-plane joint efficiency; (a) width of joint opening; (b) restraint due to direct compressive stress in the slab; (c) type and size of coarse aggregate; and (d) presence or absence of dowel bars.

Plastic theory of reinforced concrete design

CHARLES S. WHITNEY, *Proceedings*, Am. Soc. C. E., Dec. 1940, p. 1749. AUTHOR'S REVIEW

It is the purpose of this paper to present a realistic method for the design of reinforced concrete members that should result in more efficient use of the materials. The plastic theory recognizes the true characteristics of the material as determined by research since the standard "straight-line theory" was adopted about a generation ago. It has already been given recognition in the standard formula for axially loaded columns, and the formulas proposed in this paper extend the same theory and provide for the first time a consistent treatment for beams, eccentrically loaded columns and arches. The equations are much simpler than the present standard formulas and agree better with test results. They are based on the cylinder strength of the concrete and the yield strength of the steel without the use of the modular ratio, n. Portions of this theory have been published previously. ("Design of Reinforced Concrete Members under Flexure or Combined Flexure and Direct Compression," by Charles S. Whitney, JOURNAL, Amer. Concrete Inst., March-April, 1937, p. 483, and "Eccentrically Loaded Reinforced Concrete Columns" by Charles S. Whitney, *Concrete and Constructional Engineering*, London, Nov. 1938.) It has now been extended and improved so that it should form a satisfactory basis for practical design and further research.

This paper presents the theory which forms the basis of the design method suggested in the recent report of the American Concrete Institute Committee 312 on Plain and Reinforced Concrete Arches in the A. C. I. JOURNAL of September 1940, and it should be read with that report. The plastic theory was originally developed

by the writer in connection with his work on Committee 312 but was published separately because of its broad application to concrete design.

The elastic theory remains the best method for the determination of bending moments and thrusts in statically indeterminate structures but the plastic theory provides a simpler and more direct method of proportioning the members of the structure after the moments and thrusts are calculated. It will greatly facilitate the design of continuous frames and arches.

The simple method of determining the strength of a member for the full range of eccentricity from zero to infinity makes possible a more scientific design of members subjected to both bending and direct load. Proper consideration has not been given formerly to the fact that when the tensile strength controls in such a member, the bending moment that can be resisted increases with the direct load and, conversely, that it is possible for a reduction in direct load to cause failure without change in moment. The current method of design is ordinarily based on the assumption that in case of overload, both the direct thrust and the bending moment will increase proportionately, and this may not always be safe. The new method makes it possible to consider directly the probable combinations of increase in thrust and bending to which the member may be subjected. For instance in an arch rib the possibility of a great increase in thrust is remote while a considerable increase in bending moment is quite possible. It is therefore more logical and conservative to design the rib so that it can not be injured by an increase in moment if the thrust is not also increased, than it is to apply the same factor to both thrust and moment simultaneously.

The paper compares the plastic theory with the standard theory and states the reasons why it appears to be superior.

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The American Concrete Institute—How It Functions—What It Stands For

Presidential Address*

BY R. B. YOUNG

The Institute is an association of individuals, banded together "to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge." To accomplish this purpose, it has to have some form of organization, and this organization has to have a governing body.

In the Institute, the governing body is the Board of Direction. The Board, made up as it is of men giving voluntarily of their time, cannot itself, perform the multitudinous duties involved in the conduct of the Institute's business and as a matter of practical necessity, has to delegate certain of its work to others, namely, the Secretary-Treasurer, his assistants and the Administrative committees, to act as agents of the Board within the authority given them.

For the most part, the Board is an elective body; it consists of a president, two vice-presidents, secretary-treasurer, three directors-at-large, six regional directors and five past-presidents. Directors-at-large are elected, one each year, for three-year terms; the regional directors and officers for one year. This provides both continuity in management and the continual change in personnel necessary to prevent stagnation. To make the latter more certain, the by-laws provide that elective officers and regional directors may be re-elected but once to succeed themselves in the same office. The secretary-treasurer is appointed by the Board.

The perpetuation of a Board is provided for by the election each year of a nominating committee, the duty of which is to prepare a

*Presented by the retiring president at the 37th Annual Convention, Washington, D. C., Feb. 19, 1941.

ballot for submission to the membership. With the ballot is sent out a list of 20 names, also chosen by the committee, from which each member is asked to indicate his choice of five to serve with three presidents (last past) as the nominating committee for the following year.

Thus, while it might be said that members do not choose their governing Board directly, the process is truly democratic, especially so since the procedure exists with the consent of the governed and may be changed at the will of the membership. This procedure is common to most technical organizations national in scope and is the development of a necessity which arises from the fact that few members have more than a casual interest in organization management, or the time or information for participation in its detail.

Regardless of how it is governed, the Institute is an association of individuals and conversely, the members collectively are the Institute, and we should think of it in this way rather than as an impersonal organization or in terms of its management. We, the members, are the Institute and whatever it accomplishes or fails to accomplish is in some measure the responsibility of each and every one of us.

I think that it would serve a useful purpose, if, at this point we were to consider why we, as individuals, are members of the Institute. What was the urge that made us join and what keeps us paying dues year after year?

A few people join the Institute because of its prestige, a few for such reasons as to please a friend or do an associate a favor, but the majority join because of something they think they are going to get in return. Therefore, let us consider briefly what the Institute has to offer.

The principal merchandise which the Institute sells are the JOURNAL, the Annual Convention and the work of its technical committees. Since the majority of its members neither belong to its committees nor regularly attend its conventions, its most important stock in trade is the JOURNAL. On the merits of the JOURNAL more than on any one thing, rests the continued success of this organization. The JOURNAL is the tangible evidence of the Institute's many efforts to serve its members and the concrete industry.

The Institute is neither large nor wealthy and the JOURNAL is one of its major expenditures. The JOURNAL budget is limited by the income of the Institute and this in turn, is largely dependent on the dues of its members. The Institute can print each year only so many JOURNAL pages and its problem is one of serving the greatest

number of readers within the covers of a publication which it can afford to edit and print.

While the scope of the Institute's activities is confined to the narrow field of concrete, yet within this field, the interest of its members is very varied. Some are concerned with the design of concrete, some with its technology and others, its construction. To serve all members, the JOURNAL must cater continually to each of these groups, which it cannot do if its pages are filled with long papers. The thousand pages which approximately represent the JOURNAL'S present annual budget, will cover only twenty papers of fifty pages each but it will permit one hundred ten-page papers, and more if they are shorter. In one hundred papers there are sure to be several of interest to every member, in twenty there may be none. The best interests of the Institute and its members demand that the JOURNAL cater to the majority, not the few.

For some years, therefore, the emphasis has been on short, meaty papers. A paper will not be refused just because it is long, for there are many such that justify the space they require, but they are the exception and to be published have to be unusually timely and valuable. The ideal paper tells its message in as few words as clarity and the proper coverage of its subject permits and has eliminated from it all extraneous and unnecessary matter. Admittedly, this policy is hard on authors but it is easy on readers, and since it should be the author's purpose to serve the reader, not merely to gratify a desire to see his thoughts in print, he gains by brevity in the size of his audience and in his reputation.

The need for diversity in JOURNAL papers develops its own problems. Theoretically, every member's interest should be catered to in turn, but practically, this is seldom possible. The Institute has no paid or staff writers; it depends entirely on voluntary contributions. It frequently happens that there is not available a paper of the kind that is needed to supply a given reader interest, but publication schedules must be met and so some other type of paper is substituted. As a consequence, one class of reader may, for a time, find more of interest in the JOURNAL than another and if this should continue, the reader whose interests are not being met, will become dissatisfied.

This condition is often the fault of the reader group, not so well served. Some groups are more articulate than others—for instance, the research worker. Often he needs no encouragement to write papers; the very purpose of research makes him a natural author. On the other hand, it is difficult to get the average construction or field man to prepare a paper; he instinctively shies from every attempt

to have him tell publicly what he knows or has done. This is readily understood, for he is seldom a practiced writer or a regular attendant at conventions and is likely to view the whole matter with a mixture of awe and contempt; awe of the unfamiliar, and contempt for authors as program prima donnas, whose doings are not in keeping with the he-man standards of the construction fraternity.

This condition of affairs is unfortunate, for there is a crying need for papers that tell of a problem that has been solved in some novel or better way. The problem does not have to be new or intricate; the common problem which the many meet, and its solution, is of more interest to the majority than the rare problem seldom encountered. The former is not to be despised simply because it happens to be the daily companion of so many. This does not mean that the JOURNAL should become a concrete primer; a discussion of the commonplace does not necessarily involve a repetition of well-known and well-established facts and practice; it can be valuable to even the best informed student of concrete, if, in its treatment, it shows the way to a better or simpler solution of an old problem.

Another type of paper that has a wide appeal is the one that puts the results of research into a form usable by the practicing engineer or architect. It, more than any one thing, cuts down the lag, so often complained of, between research and practice. The need for such papers is great, for the busy practitioner has little time to delve through the literature of concrete research, sift data, weigh conclusions and determine their significance. He needs to have this done for him, so that he has only to work out the methods for applying the new information to his particular problem. To write interpretative papers successfully, a man must have a good knowledge of concrete theory and practice, of laboratory and field methods, and must combine with these, the ability to write clearly, simply and convincingly. Unfortunately, few men possess these qualifications.

As a matter of information, it might be well to outline here, the procedure followed by the Institute in handling a paper, report or discussion offered for publication. No paper is accepted that does not meet certain minimum requirements. Besides the usual standards of decency and good taste, it must fall within the field of the Institute's activities, its substance must add to our knowledge and be worthy of permanent record; it must be the original work of its authors and not have been published previously and it must be as concise as possible and reasonably well written.

The enforcement of these standards is the duty of the Publications Committee which passes upon every paper, report or discussion that

appears in the JOURNAL. Each paper is first read by several competent critics who report to the Publications Committee whether or not, in their opinion, it is acceptable and what might be done to improve it. If the subject meets the Institute's standards with respect to substance and originality, every reasonable attempt is made to assist the author in the matter of length and form. Few contributions to the JOURNAL are published as originally submitted, the majority receive extensive editing; and while most authors submit to this process with some misgivings and an occasional protest, in general they realize its necessity and cooperate cheerfully.

The second service which the Institute offers its members is the Annual Convention. The Convention serves as a forum and a place of meeting. As a forum, it affords an opportunity for the presentation and discussion of papers and reports. As a place of meeting, it offers comradeship, the opportunity to meet others in the industry and with them to exchange ideas, trade information and test ones theories against their experience. The meetings of the Institute are very vital and informative as all who have attended them can truthfully testify.

The work of its technical committees is the third service that the Institute offers in return for membership. Just as it can be truthfully said that an individual cannot claim to have fulfilled his responsibilities as a citizen, unless he has given in return for its privileges, some measure of service to his fellow men, so an organization such as the Institute is not fulfilling its responsibilities if it exists only to serve its members. The Institute, through its technical committees, serves the industry of which it forms a part and in a larger sense, the public. Only by such service does it justify its existence.

Like most service, the work of the technical committees also benefits those who perform it. A member with an active interest in the organization is a loyal member and therefore the more persons working on committees, the better for the Institute, provided, of course, there is worthwhile work to be done. However, there seems to be no limit to the work that could profitably be undertaken, the difficulty is to find men qualified to do it.

The qualifications for a good committee member are a specialized knowledge in the field of the committee, time to devote to its activities and a willingness to serve. The best committee member is one to whom the task assigned is part of his daily job and who has a pressing and personal interest in its solution, for there is nothing like the bread and butter motive, in this as in other things, to make a man work.

Few men will refuse committee assignments but only a limited number really function; most of a committee's work is done by a minority of its members. The blame for this condition lies partly with the management of the Institute and partly with the individual member. Management's principal fault is the tendency to overwork the willing and the conscientious. In choosing committee personnel, the mistake is often made of assigning a good worker too many committees, for it is always easier to do this than to discover new talent. But a man can only do so much and no matter how willing he may be, he cannot perform at his best if burdened to the point where the task becomes a chore instead of a pleasure.

No person has any right to accept membership on a committee who is not able or willing to contribute to its work. The bane of all committees is the non-functioning member; he slows down progress and adds to the burden of the others. Non-performance may be due to various reasons. There is the person who accepts membership because of the prestige it brings him and who feels under no obligation to do anything in return; fortunately, he is in the minority. With most non-performers it is a question either of time or interest and of the two, the more important is interest, for some of the best committee workers are very busy men yet they seem to manage. Most people will find time for those activities that interest them, and Institute members are no exception.

Since service to the Institute is voluntary, the individual member cannot be driven in his task to the same extent as where service is paid for. The result is that many jobs, which should be done quickly, lag and many are not done at all. This is inevitable in an organization such as ours and must be reckoned with at all times. The wonder to the writer is that so much that is worthwhile can be done purely by voluntary help; that so many are willing to give of their time and talents to the work of the Institute.

Committee work is organized in various ways. In the Institute, the only standing committees are administrative; all others are appointed to do a specific job and when that job is done, they are discharged with thanks. It has been found from experience, that for best results, committee assignments have to be definite, limited in scope and capable of accomplishment within a reasonable time. For instance, to assign a committee the entire field of aggregates is to give it so much territory in which to work that it is overwhelmed by the magnitude of its task and ends up by doing nothing. But ask it to study one phase of the aggregate problem, such as the safe limits for shale, and it has something definite to do and does it.

The functions performed by technical committees might be classified as discovery, assembly, interpretation and codification. These four functions cover the several steps from research to standards. Discovery of pertinent facts forms the basis of all technical progress, the assembly of those facts and their interpretation are the intermediate steps, and the final is the preparation for the guidance of practitioners of specifications or standards of practice. The Institute has committees working in all four fields.

Many committee projects of the first type, i.e., for discovery, lag or are impossible because of the inability to develop needed data; experimental research is required on which to base a report. The Institute would like to help committees financially but cannot do so within its present income. Occasionally, but only occasionally, as in the case of the column tests of a few years ago, is it justified in soliciting from industry, the funds necessary for such work. Usually the best that it can do in the field of research, is to point out problems that are pressing, encourage men or institutions to undertake them and offer through the pages of the JOURNAL the means of publicizing the results obtained.

One of the most important of the committee functions is the preparation of standards in the form of specifications and recommended practices. Standards are end-products of all committee work, and they represent the crystallized knowledge of research, clarified and refined in the minds of many skilled men to form a guide for performance. They become a part of the contractual structure of our industry and are probably the most important documents that the Institute issues.

Because of the importance of standards, certain safeguards have to be set up to see that none are issued that are not worthy. All specifications and recommended practices have to be reviewed critically by the Standards Committee, one of the Institute's administrative committees. The Standards Committee does not pass upon their technical features but it does see that the documents are complete, in proper form and consistent with the Institute's other standards. It also makes sure that all points of view have been considered by the sponsoring committee and, in the case of any marked differences of opinion within that committee, it requires, that these differences be composed before the standard is presented to the Institute for discussion. If it then passes the critical eyes of the Institute's members, it is voted upon at a convention and by letter ballot and if approved, then becomes an Institute standard. On the other hand, if serious criticism of it develops, it is sent back to its originating

committee for further study. A standard, once adopted, has to be reviewed every few years to keep it up-to-date. In these ways, the standards of the Institute are maintained at a high technical level and the user can be assured that each represents the best available thought and knowledge in the field it covers.

In preceding paragraphs, brief reference has been made to the inability of the Institute financially to expand its publications greatly or to underwrite experimental research. Many other needed services could be rendered its members and industry, if the Institute had the money to spend but they have to await more income. One Board member has likened the situation to a circle, more members, more income; more income, more service; more service, more members; and so on and on. The problem of Institute expansion is where can one break into this circle?

I am not going to attempt a solution of this problem. The truth is that I do not know the solution and seriously question that there is one ready at hand that could perform the miracle of rapid expansion of the Institute's membership and income. Further, I am not at all sure that rapid expansion would be desirable; the strongest organizations are built slowly and the Institute might be the loser by a too rapid growth. But some growth is essential to existence and so the Institute must strive to enlarge its usefulness if it is to survive in these trying times.

In the foregoing discussion of Institute affairs, I have tried to picture to our members what we do to justify our existence, what we offer in fellowship and service to our members and how we guard our honor and prestige. To me these suggest ways in which the Institute's work may be strengthened, ways that are not spectacular but are sound, i.e., more and better papers, more and better standards, and more interesting meetings. All these are possible within our present financial resources. With these, it will be increasingly easy to gain new members, to increase the sale of our publications, to obtain the support of industry. In such a program every member can help and I trust, will help. If he does, the Institute will prosper; it will gain in influence and prestige and become a truly great organization, if not in size, at least in the values that count.

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Consistent Inconsistencies in the Consistency of Concrete*

By C. H. Scholer†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Concrete segregation, usually considered objectionable, is to some extent necessary and desirable, as in placing and finishing operations at surfaces and around reinforcing. Concrete should be designed accordingly.

"Bleeding" of concrete, long recognized as objectionable, has its uses. Construction practice depends upon free water for lubrication. Arbitrary reduction of bleeding may lead to serious errors.

Density of concrete is a common measure of its quality. Concrete technicians know that the densest concrete is not obtained with rich mixtures yet density is commonly considered a result of rich mixtures. The use of cement admixtures to promote workability actually reduces density by incorporating air, but nevertheless improves durability.

INTRODUCTION

In his recent book, "As I Remember Him," an autobiography by Dr. Hans Zinnser, the late Dr. Zinnser presents a most interesting account of the difficulties experienced in securing the adoption, by the medical profession, of the latest scientific facts and procedures. The author was particularly interested in his description of the underbrush of misinterpretation and misinformation that had to be cleared out before a proper understanding of many phases of the problem could be attained.

The present status of the engineering profession, in its appreciation of some of the most common characteristics of concrete, as well

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†Head, Dept. of Applied Mechanics, Kansas State College, Manhattan, Kansas.

as more recent developments in concrete quality control, prompted this paper and its title.

Some of the densest, most thorny thickets of underbrush are clustered around our thoughts on segregation, bleeding, and durability. Let us get our hands scratched trying to clear this out.

Concrete segregation, usually considered an objectionable characteristic of concrete, is, to some extent, a necessary and desirable property of this material. All placing and finishing operations involve the production of some segregation at finished surfaces and around reinforcing bars. Our failure so to design our concrete mixtures that the minimum of the desired segregation, necessary for finishing and placing, may be secured easily has frequently lead to unsatisfactory results, particularly in thin slabs and pavements. The presence in such construction of over-sized, coarse aggregate frequently leads to the use of excessive slumps and objectionable, undesirable segregation. This paper discusses some of the factors influencing segregation and suggests a unique method for determining the maximum permissible size of coarse aggregate to be used in a given type of construction.

The phenomenon of bleeding of concrete has long been recognized as an objectionable characteristic. Bleeding, however, has its uses, and much of our construction practice depends upon the presence of free water to lubricate the finishing tools. The arbitrary reduction of the bleeding tendency by the use of finely ground cement, or any means except the reduction in total amount of water used, may lead to serious errors in construction practices. The paper points out the widely varying characteristics of concrete mixtures made with cements having markedly different bleeding characteristics.

The density of concrete has frequently been used as a measure of concrete quality. Generally, the assumption is made that the quality improves with higher density. While all concrete technicians recognize that the densest concrete is never a rich mixture, those who speak most frequently of density think of the terms as being in agreement; that is, that density implies a rich mix. Most engineers have thought of density in terms of the maximum density for a given arbitrary quantity of cement, sand and stone. The maximum density for a fixed mix can be achieved only with the proper water content which is usually considerably less than is being used. Thus it can be seen that our concept of high density and related durability has really been a concept of low water content.

Admixtures with cement—admixtures which promote increased workability with lower water content, but which also incorporate air

in the finished concrete (thus reducing the density of the concrete) are efficient in improving durability, not because of the lack of density, but because of the elimination of the water which otherwise would be in the mixture. It is thus seen that the increased durability with these treated cements is not in contradiction to our original concept of increased durability with increased density.

SEGREGATION

Segregation is, of course, objectionable. Engineers and builders attempt to design mixes, and use such consistency of concrete and methods of handling and placing the plastic mass, that the minimum separation of their materials will occur.

Those are desirable objectives; but we must at the same time recognize that we desire and must have segregation. Some segregation must occur at every concrete surface, at all side forms, and around reinforcing. At all finished surfaces, we demand and secure by our finishing and placing operations at least this minimum of segregation. If the required segregation is considerable, and difficult to secure, it is almost certain, in the hands of the average builder, to result in the use of excessive slumps in the concrete mixture and probably serious segregation.

The amount of segregation required to produce the desired finish at a surface is a function of the maximum size of the coarse aggregate, the uniformity of grading of the coarse aggregate and the relative amount of mortar in the mix. The large pieces of aggregate will have to be moved back from the surface displacing larger volumes of mortar, producing more segregation than would occur if the aggregates were smaller. If well graded coarse aggregate is used, the percentage of large pieces will be smaller and less movement and less segregation will be required at the surfaces. The larger the relative volume of mortar the less frequently will coarse aggregate have to be moved back.

The ease with which this segregation can be produced is again influenced by the size of the coarse aggregate, the uniformity of grading of the coarse aggregate, and the relative amount and consistency of the mortar in the mix. In addition, the ease of segregation is influenced very largely by the relation of total surface produced to the volume of the concrete adjacent to these surfaces.

The relation between the size of aggregate, and the ease of segregation is obvious. The effect of uniform, dense grading is to make it more difficult to produce segregation, but as mentioned above, it

reduces the amount of segregation required. The use of well graded coarse aggregate of large maximum size will tend to reduce the relative mortar content of a well designed concrete mixture if designed for strength and economy. The large maximum size increases the segregation required and the decrease in mortar content makes the problem of producing the *necessary* segregation more difficult. If the concrete is placed in sections requiring a considerable surface area to be finished or contains large amounts of reinforcing the design should be modified to decrease the required amount of segregation and to make it practicable to produce this necessary minimum of segregation.

Some reduction in maximum size of aggregate is usually provided for reinforced concrete structures in which reinforcing details make such provision necessary. The chief type of construction where this problem has been almost entirely ignored has been in thin slabs, particularly concrete pavement construction. The relatively large ratio of surface to volume in a concrete pavement, and the necessity for a finish which will insure good riding qualities would indicate the need for the design of a concrete mixture especially suited for this condition. The oversize stone so frequently permitted in a concrete pavement seriously complicates the placing of concrete particularly at joints and along the edges. In some of the recent designs of load transfer joints the problem becomes exceedingly difficult. The tendency is to add excess water or design a mix that can be handled easily at the joints because of its high mortar content, and high slump. This concrete will then have a tendency to segregate readily throughout the whole pavement and result in thick mortar coats over the coarse aggregate, high water content in the upper layers of the surface, and lowered durability.

This problem should be met by the design of concrete mixtures requiring a minimum of segregation at the surfaces and joints by choosing a smaller maximum size coarse aggregate, well graded and with but little if any increase in mortar content, and with less fluid consistency.

For some time the author has given thought to the possibility of developing a sound empirical method for the determination of the maximum permissible size of coarse aggregate considered as a problem in segregation required for proper finish. This value might be called the radius of segregation. Like its counterpart in hydraulic practice, it would be represented by $\frac{\text{the volume}}{\text{surface area}^*}$. It would repre-

*The area of all surfaces produced, including reinforcing bars, dowels, and other joint surfaces.

sent the average distance from any point to the nearest surface which required the rearrangement of aggregate particles and consequent change in relative mortar content. For members of uniform section and with great length compared to other dimensions, the value can be found by dividing the area of cross section by the perimeter of all surfaces.

For a typical pavement slab 8 in. deep and 11 ft. wide this becomes, neglecting reinforcing:

$$\text{Radius of Segregation} = \left\{ \frac{8 \times 11 \times 12}{2 \times 11 \times 12 + 16} \right. = 3.77.$$

If a thickened edge section is used and a considerable portion of the slab is reduced to 6-in. thickness, the value is reduced to 2.9 in. Obviously, the problem of finishing these two slabs and properly compacting the joints will be more difficult for the 6-in. slab if coarse aggregate suitable for the 8-in. pavement is used with the 6-in. pavement.

It is suggested that a suitable value for the maximum size of coarse aggregate would be not more than 55 per cent R. S. (radius of segregation). The author has only limited data as to just what figure should be used for this factor. He would like to point out that many of the famous old concrete roads he has had the privilege to examine were built with a much smaller limiting size.*

Failure to recognize the need for a reduction in amount of segregation required causes many troubles. Joints have always been a source of weakness in concrete pavements. To overcome the difficulties encountered in placing concrete at joints there is a tendency to over-water the mix, to use special puddling and manipulation to produce the desired segregation. There is now a general tendency to place the concrete at joints with a vibrator. If the rest of the concrete is to be placed without vibration the consistency of the concrete will be such that there is great danger of over vibration at the joint.

After all, a vibrator is merely a device to promote segregation. It should be used only on concrete of such consistency that segregation is not easily obtained. Its use in concrete designed to require a minimum of segregation difficult to produce should be advantageous.

The concept of radius of segregation and its significance in complicating the placement of concrete can be applied to some of our testing problems. In making a slump test we use a standard slump cone. In properly compacting this concrete with large size aggregate considerable segregation, top, bottom, and sides must be produced.

*Includes the Ohio Post Road; 30-year old roads in Wayne County, Michigan; a number of 26-year old gravel concrete pavements in Pittsburgh, Pa.

The value of R. S. for the slump cone is 1.2 in. This will require much rearrangement of relative mortar content in the mass of plastic concrete if the concrete contains large size aggregate. How well does the workability of concrete made with 2-in. aggregate compare with that of concrete made with 1-in. maximum size aggregate when both concretes have the same slump? Six by six by 36-in. beams have a value of R. S. = 1.38 in.; for a 6 by 12-in. cylinder, 1.2 in. It has been found necessary to limit the maximum permissible size of aggregate to be used in these commonly required sizes of test specimens. Proper limitation of R. S. might be the best way in which to state such limits.

BLEEDING OF CONCRETE

Bleeding, long recognized as an objectionable characteristic of concrete, has its uses, and much of our construction practice depends upon the presence of free water for lubrication of the finishing tools. The arbitrary reduction of the bleeding tendency by the use of finely ground cement or any means except the reduction in total amount of water used may lead to serious errors in construction practices.

Probably the most significant paper on concrete presented in many years is T. C. Powers' paper "The Bleeding of Portland Cement, Paste, Mortar and Concrete" appearing in the A. C. I. JOURNAL for June, 1939. This paper and Bulletin No. 2 of the Portland Cement Association of which it is a condensation should be carefully studied by any one interested in concrete placing and handling problems.

Mr. Powers' paper presents a well planned, scientific investigation of the whole phenomenon of sedimentation in a mass of plastic concrete. We cannot review all of it here, but it is desired to call the attention of engineers, particularly those engaged in concrete pavement construction, to a slightly different aspect of the problem than is presented in detail by Mr. Powers.

It has long been considered by engineers that if concrete mixed with a certain water-cement ratio was placed in the forms or on the subgrade that the resulting concrete was concrete with this water ratio. This is, of course, incorrect. Immediately the concrete is deposited consolidation begins to take place, the heavier ingredients displacing the water which travels upwards through the capillary structure of the mass. Mr. Powers' paper would indicate that if this concrete were placed and then allowed to remain undisturbed by placing and finishing operations, that the water ratio of the upper layer would remain substantially at the basic water ratio of the mix. The lower layers would have a decreased water ratio due to the loss

by bleeding, but there would be no tendency appreciably to increase the water content of the top, free water coming to the surface, without increasing the water content of the adjacent paste.

If, now, we introduce placing and finishing operations, we can see at once, that we will reincorporate in the upper layers this water lost from the portion below. According to Mr. Powers' figures, not more than one-third of this excess so incorporated can then be lost by bleeding. This indicates the extreme importance of so correlating our placing and finishing operations that there is as little manipulation of the concrete as possible during this bleeding period. To do this the concrete should be placed and struck off to the proper elevation as soon as possible, and no further handling or finishing done until the latest possible time. The excess water, if any, should then be removed and the desired finish given to the surface.

There is every reason to believe that many of the bad results believed to have been due to overfinishing, late finishing, excess water, poor cement or other materials, etc., were really caused by the continued manipulation during the bleeding period, and the resultant high water content of the upper layers of the concrete.

The logic of these conclusions is applicable to all types of construction. One has only to look at structures exposed to severe weathering conditions. The top of retaining walls, and top of intermediate construction joints or lifts, bridge parapets, pier caps, hub guards, curbs, are all points where early trouble develops. This inferior concrete was produced by our finishing and manipulation processes which reincorporated in the upper layers the water lost by bleeding from the lower layers.

We do not see this condition in some of the worst cases because, generally, the worst cases are to be found in building construction and are not exposed to the weather. About two years ago the author had an opportunity to see what happens to some of the worst of these practices when he examined a building frame erected during the depression and allowed to stand unprotected for several years. The disintegration and deterioration was extremely serious.

These detrimental effects of bleeding have led to the adoption of many procedures to reduce bleeding. The use of admixtures, more fines in the sand, less water in the concrete, and the use of finely ground or special cements. All of these devices have been tried and each has been found effective in certain instances.

The most obvious and best device is to use less water in the concrete. For a given mix the bleeding is directly proportional to the amount of water in the mix. Any reduction in bleeding effected by

reduction in the total water content of the freshly mixed concrete is advantageous. In fact, it is reasonably certain that in instances where bleeding has been reported as decreased by the addition of fines to the sand or a better grading of the sand, that the real reduction in bleeding was secured by a reduction of the total water in the concrete; the increased workability of the concrete by the improved grading of the aggregate making this possible. Mr. Powers' paper indicates that the addition of fine sand (unless of a fineness comparable to cement) would have a negligible effect upon the bleeding tendency of the concrete, except as it might decrease the water required for workability.

Tests by Gruenwold* indicate that where enough such fine sand has been added to retain the water and really decrease the bleeding tendency, the resulting concrete shows decreased durability.

The finer the cement is ground the lower the bleeding rate and the bleeding capacity for a given water content of the concrete. This characteristic of a concrete made with a finely ground cement is probably of some advantage in certain types of construction. The author is of the opinion that it also offers serious objections in many instances. These objections will be discussed later.

Certain admixtures in concrete or in a cement are effective in reducing the bleeding tendency.

Most of these admixtures are of the type which cause the incorporation of air in the cement water paste. This air is very effective in promoting workability and reducing the amount of water and paste required for a given consistency. Any reduction in bleeding secured by reducing the total water in the concrete is desirable. It should, however, be pointed out that the properties which make possible this reduction in water content may also reduce the compressive strength of the concrete. A balance between these features should be secured in the design of the mix.

There are practical limits to the possible reduction in total water content of a given mix. The necessity for handling and placing the concrete will frequently result in a higher bleeding rate than seems desirable. This is particularly likely to occur where rather deep sections of concrete are being placed. When this occurs the reduction in the bleeding rate by the use of admixtures or finely ground cements may possibly be desirable. Again it should be pointed out, however, that if these admixtures or cements with high water retentive capacities are used, precautions must be taken to prevent the use of excessive quantities of water and also to see that the placing and finishing

*"Lean Concrete Mixes"—Ernest Gruenwold, A. S. T. M. Proc., Vol. 39, 1939.

processes do not result in incorporating in the upper layer of concrete the excess water coming up from the lower layers. Where cements or admixtures with high water retentive capacity are used this excessive water will not be evidenced by the accumulation of free water on top of the concrete. Where possible, the water on top of the concrete in deep sections should be removed or compensation made in the succeeding batches of concrete to allow for the excess water, bleeding from the portions below.

USES OF BLEEDING

Most of the preceding discussion was concerned with the objectional features of bleeding and how these might be controlled. We should, at the same time, recognize that for most of our plastic concrete some bleeding is required. In thin horizontal slabs and in pavement construction, the appearance of free water on the top of the concrete is frequently used as a guide for the required amount of water to be added to the mix. The top of the concrete must be wet to permit ordinary finishing procedure on thin slabs and in concrete pavement construction. When, due to low water content, or to cement and aggregate characteristics having low bleeding rates, the top of the pavement does not show some free water, it is customary in most construction practices to increase the basic water content of the mix. The author has noted this in many instances where high-early-strength cements were being used. The low bleeding rate of concretes made with such cements frequently requires the addition of considerable quantities of water to the mix, far above the requirements for workability in placing, so that the surface would not be too sticky and difficult to finish by the ordinary procedure. Frequently engineers think of this kind of concrete as having set up too fast and are inclined to blame this characteristic on the fact that high-early-strength cement sets too rapidly. This is not the case; such surfaces sometimes are very sticky and almost dry, while the mass as a whole is of a jelly-like consistency. Increases in water content of as high as one gallon per bag of cement have been noted in some instances where finely ground, high-early-strength cement was being used.

Let us consider the significance of cement fineness on the bleeding rate of a concrete. Take as an example the same concrete mix used by Mr. Powers (Page 63, PCA Bulletin 2). In this example he used cement 14502—a typical modern cement with fineness of 1860 sq. cm./grm. (see table, p. 34, Bulletin 2).

Using this cement, it is found that this concrete has a bleeding rate of $Q = 35 \times 10^{6-}$ cm./sec., and a slump of 4.7 in.; the water content was approximately 4.6 gal. per sack.

Now replace cement No. 14502 with No. 14560, a cement ground to what was no doubt a satisfactory fineness with the standard practice of 20 years ago. The cement has a specific surface of 1085 sq. cm./grm. By means of equations 226 and 14 of Bulletin No. 2, using exactly the same quantities of cement, water and aggregates, it will be found that the bleeding rate is: $Q = 167 \times 10^{-6}$ cm./sec. —more than 4.5 times the bleeding rate of the first concrete. Similarly, computing the bleeding rate, if cement K20 (a high-early-strength cement of 2540 specific surface area) were used, one would secure a value of $Q = 12.4 \times 10^{-6}$ cm./sec., only one-third of the bleeding rate of a normal cement, one-fourteenth that of the old-fashioned cement. To bring the high-early-strength cement concrete up to the same rate of bleeding as the normal cement would require 28 per cent more water, an increase of about 9 gallons per cubic yard. To produce concrete made with this finely ground cement with the same bleeding rate as the old-fashioned cement would require 74 per cent more water, an increase of more than 20 gallons per cubic yard. This would obviously produce a mix of such consistency that bad segregation would result, and the equation would not apply. Practically, it would not be possible to produce a concrete using such finely ground cement that would have the same characteristics as when the coarse ground cement was used.

To try to produce concrete with the old-fashioned cement with a bleeding rate of only 35×10^{-6} cm./sec. (that shown by the typical modern cement) would require a reduction in water content of more than 25 per cent. The resultant mix would not be usable if the original concrete with cement No. 14502 was a suitable mix.

In one instance, in the laboratory where a concrete pavement mix was being placed with three different cements, it was found that using the same aggregates to produce the same bleeding rate and apparent wetness of surface for finishing would require for Cement "A" at 3-in. slump, 4.8 gal. of water per sack; Cement "B" required 5.8 gal. of water per sack, and Cement "C," a finely ground high-early-strength cement, required 6.9 gal. of water per sack to give the same apparent finishing characteristics.

Concrete pavement placed with cements having high water retentive capacity and low bleeding rates may be badly abused if the engineer does not recognize that he always requires some bleeding, and that concrete which does not bleed may be very defective if

enough water is added to make it bleed in accordance with his ideas of the desired wetness of surface. Where a concrete in thin slabs bleeds so slowly that a wetness of surface for finishing is not secured, water should be added by means of a fine spray. This condition is not commonly met with except in warm weather on drying days, or where unusually fine cements or admixtures are being used. Failure to recognize these conditions when they have developed has led to the construction of much inferior concrete. The surface characteristics of some sands tends to the development of premature bridging and a consequent checking of the bleeding rate. Some of these sands may require water varying from 10 to 20 per cent in excess of that required with other sands using the same cement. When such sands are combined with cements or admixtures producing concrete with high water retentive capacity the concrete is likely to be badly abused.

To reduce the bleeding rate of concrete by any means other than the reduction in the total water content of the concrete is a hazardous procedure. Cement and concrete technicians have gradually introduced into use cements and aggregates having distinctly different characteristics from those of not many years ago. We have not warned the ordinary user of these changing characteristics of the concrete when these cements are used. In most instances the average contractor or builder, and alas, too many engineers, are still using the criterion of 20 years ago for the proper consistency and wetness of surface for a concrete mixture. The result is that many engineers and builders think they are using much drier concrete than they used in their earlier experiences, when as a matter of fact, they have increased the water content considerably. Concrete is deserving of much more careful and thorough study than it has had during recent years. We have had very exhaustive (or exhausting) studies of cement, sand, stone, and the design of concrete mixtures but little thorough study of the real characteristics of a concrete mixture. Mr. Powers' paper leads the way toward such a critical study.

DENSITY

Another item on which we engineers have been consistently inconsistent is related to the density of concrete.

The density of concrete has frequently been used as a measure of concrete quality. Generally, the assumption is made that the quality improves with higher density. While all concrete technicians recognize that the densest concrete is never a rich mixture, those who speak most frequently of density think of the terms as being in agreement; that is, that density implies a rich mix. Most engineers have

thought of density in terms of the maximum density for a given arbitrary quantity of cement, sand and stone. The maximum density for a fixed mix can be achieved only with the proper water content which is usually considerably less than is being used. Thus it can be seen that our concept of high density and related durability has really been a concept of low water content. The author believes that experience with such concretes and such variations in density has given rise to the vogue for the worship of dense concrete. When we recognize that most of the value of increased density in a mix of fixed proportions is due to decreased water content per unit volume of concrete we are not so surprised when we find that the use of admixtures in the concrete or in the cement, which incorporate air in the paste thus reducing the density of the concrete, usually give a remarkable increase in the resistance of such concrete to freezing and thawing. The incorporation of a small quantity of air in this paste promotes workability and plasticity more effectively than the incorporation of the same amount of water would have done. The result is that for the same workability the total water content has been materially decreased with the resultant increase in resistance to the disintegrating effect of freezing and thawing.

Without question, the resistance of this concrete to disintegration is further increased by the presence of these air voids. It is common knowledge that a low saturation coefficient in most masonry materials increases their resistance to disintegration by frost action. Presumably this increased resistance is due to the fact that as water freezes and pressure tends to develop, it is crowded into these small air voids without the development of destructive forces. While experimental data confirming this explanation of the increased resistance of concretes made with treated cements which do incorporate these air voids is still lacking, one would expect our experience with other types of materials to be repeated in this case. Observations of such concrete after a service of many years, and innumerable laboratory tests confirm the advantages of such concrete; the disadvantages of lowered density and consequent small loss in strength being more than offset by the gain due to decreased water content per unit volume and lowered saturation coefficient.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the JOURNAL Supplement concluding this Proceedings volume

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Discussion of a Paper by C. H. Scholer:

Consistent Inconsistencies in the Consistency of Concrete*

BY ERNST GRUENWALD AND C. H. FORESMAN

BY MR. GRUENWALD†

Professor Scholer's observations coincide very closely with our own experience. His paper provides a valuable link between laboratory research and field practice. Laboratory tests make it possible to study the fundamentals of concrete under a given set of conditions, but it is difficult to reproduce the wide range of field conditions, particularly as regards placing and finishing. The paper offers a definite contribution on the problems of segregation and bleeding.

Contrary to the accepted belief that bleeding is undesirable, Professor Scholer presents the viewpoint that to some extent bleeding is a necessary and desirable property of concrete. His remarks refer in the main to paving concrete, where the presence of free water is necessary to expedite finishing, which usually starts shortly after the concrete is deposited.

We know that the fineness of the cement affects bleeding, and Professor Scholer's remarks on finely ground cements and high early strength cements bear directly on this point.

As we see the problem, there is a difference in the character of bleeding. In mixes lacking in fines, water alone comes to the surface. This water can be removed and this type of bleeding will generally not lead to an increase in the water ratio of the concrete near the surface. When mixes become more plastic, either through an increase in cement

*ACI JOURNAL, Apr. 1941; *Proceedings* V. 37, p. 537.
†Chief Engineer, "Incor" Division, Lone Star Cement Corp.

content or use of finely ground cements with a higher water retentive capacity, the manipulation of concrete will bring mortar, rather than water to the top. This mortar has a high water ratio, as well as a high cement content and after hardening may easily separate and produce scaling in service. This type of mortar bleeding can be avoided in the field by the precautions suggested by Professor Scholer, such as low water ratio and minimum manipulation while the concrete still is in a plastic state. However, present field operations are geared to rapid progress and in general the temptation will be to improve the finishing characteristics by adding more water, especially when extremely fine cements are used.

That raises this question: should not the Tentative Cement Specifications which now contain a minimum fineness requirement, also perhaps require maximum limits of fineness for the different types of cement? And if so, what should these limits be?

Our experience with high-early-strength cements during the last 14 years has indicated that in mixes of moderate cement content, these cements will produce a more plastic concrete and still bleed sufficiently to facilitate finishing operations. It is only in the richer mixes, containing more than 6½ bags cement that this troublesome and objectionable stickiness appears which leads to the addition of water in order to promote finishing operations on the job. This is particularly true when cements of extremely high finenesses are used.

So far as finer cements are concerned, laboratory results and field experience are sometimes at variance. In the laboratory an increase in cement fineness is usually accompanied by greater strengths and somewhat increased durability. In the field, however, under present practices of manipulation and finishing, these advantages often fail to materialize. The reason lies primarily in a misconception as to workability. In the field, workability is usually judged by the slump test, but this test does not indicate true workability, particularly with the finer cements, such as high-early-strength cements, which produce a more plastic and cohesive concrete. It will be found that for equal slumps, fine cements produce concrete of greater mobility and placeability than coarse cements. This suggests the use of lower slumps with fine cements, and thus by maintaining normal water ratios the value of high-early-strength cements can be fully realized in the field.

Professor Scholer's paper is timely and helpful, and the application of his recommendations in the field should lead to better and more lasting concrete.

BY MR. FORESMAN*

I wonder if Mr. Scholer enjoyed writing his paper on "Consistent Inconsistencies . . ." as much as I enjoyed reading it. The cement finishers could do no less than present him with a life membership in their union. Anyone who presents a gang of cement finishers with a nice "bloody" concrete that requires them to sit peacefully by for hours before they can begin their work and then provides a nice film of laitance for them to trowel easily to a beautifully smooth surface is certainly deserving of their deepest gratitude. Oh, yes, this concrete is also to be put through a process of segregation immediately as it is placed so that the finisher is assured that he will encounter no difficulties whatever in his eighteen hours of—collecting his wages. So much for fun—with apologies.

Seriously, Mr. Scholer's paper provides plenty of food for thought. A discussion of the actual subject matter can be limited to a few brief comments: First, a properly designed mix does not bleed excessively and it does not need artificial segregation, other than that produced by adequate screeding, to provide a surface that can be floated and finished. Second, Mr. Scholer in his condemnation of "density" has shown at least one way of obtaining a good dense cement paste, which is the basis of durability. By providing incorporated air for lubrication he reduces the total water and provides a more dense paste. It is possible that the air incorporated in the mix could be replaced with a hard, durable mineral. The result would be much the same. Water would be displaced and the cement paste would be more dense and more durable.

In connection with that troublesome phenomenon known as "bleeding" (and not discounting the value of the work of Mr. Powers, Mr. Sprague and others), has it not occurred to anyone that the retention of water in a concrete mix depends mainly on the surface tension of the mixing liquid (usually water) and the surface area of the cement and aggregate? Suppose we state it this way: "Other conditions being the same, the bleeding of a concrete mix varies inversely with the specific surface area of the combined cement and aggregate." It is also true that, given a constant specific surface area, etc., the amount of bleeding can be varied by changing the surface tension of the mixing liquid. Mr. Powers mentions sodium chloride. Calcium chloride will also reduce bleeding. Both of these are soluble salts and increase the surface tension of the mixing liquid. The use of soluble salts is somewhat risky because they are generally active chemically. With the

*Concrete Engineer, Board of Public Education, Pittsburgh.

exception of calcium chloride, the action of which is fairly well known, any active chemical would be subject to a rather long and involved series of tests before its use could be considered. On the other hand many inert mineral flours can be safely used. Any job where the strength and durability of the concrete is of importance should have preliminary tests made of the concrete produced by the materials proposed to be used and materials considered for control of bleeding should be investigated in these tests. Materials other than those which are entirely inert may be used when the user has reliable information concerning the action when blended with portland cement.

I would like to call attention to two points which are not contained in the subject matter of Mr. Scholer's paper but which are emphasized by his discussion. The first of these is the multiplicity of meaning attached to the various terms used in connection with concrete. As an example let us take the word "density." The term can be applied to the specific gravity of the individual aggregate particles. We have again, the density of the mass of aggregate particles. This is strictly a function of the grading and shape of the aggregate particles. Then there is the apparent density of the cement paste (reduced by Mr. Scholer through incorporation of air). The actual density of the paste surrounding Mr. Scholer's air bubbles or particles of aggregate is something else again. Finally, the density of the mass of the concrete which is a combination of all the other densities. What does one mean by density? Concrete terminology may not be worse than that applied to other subjects but it is certainly in need of revision and simplification.

The other point brought out by Mr. Scholer's discussion is the appalling lack of ordinary intelligence in much of the design and control of concrete mixtures. If the hit-or-miss methods used in concrete work were applied to the manufacture of steel I wonder what would happen to some of our skyscrapers and bridges. It may be complimentary to concrete as a building material that it has stood up at all but the methods used certainly do not reflect much glory on the intelligence of the engineers who use it.

I do not quarrel with any person's idea of what the composition of a batch of concrete should be. I may think he is wrong and certainly he is entitled to think I am wrong but there can be no question that the concrete in any section of a structure that acts as a unit should be uniform. Current specifications and present methods of design and control do not make for that uniformity. To illustrate: The current A.S.T.M. specification for 1½-in. to No. 4 aggregate sets up the following limiting percentages passing the sieves listed:

2-in.	100
1½-in.	95-100
¾-in.	35-70
⅜-in.	10-30
No. 4	0-5

Material complying with this specification may have 65 per cent between ½ and ¾-in. or it may have only 25 per cent. These extremes represent totally different gradings and if fixed quantities of material are maintained in the mix a wide difference in the void content of the combined aggregate will obtain. Any increase in the void content of the combined aggregate simply means more water to maintain the proper consistency. The result is obvious, yet how many engineers or architects require (or permit) adjustment or redesign of the mix to take care of variation of the grading of aggregates within the specification.

Specifications for concrete aggregates have been and will continue to be dependent on what the aggregate industry is able to produce with the equipment available. Grading limits must be fairly wide but it does not follow that primary variations are acceptable in the final product—concrete. It is the function of the concrete engineer to adjust variations to his needs—not accept them as inevitable. An examination of specifications and procedures for the manufacture of concrete will show, however, that this last is just exactly what the concrete industry has done and continues to do. The Joint Committee in Paragraph 305 of its report sets up certain suggested mixes. The only reference to what is desired in the grading of the combined aggregates is a foot note that the amount of sand may have to be varied to secure proper workability. The variations in the original aggregate are accepted as inevitable in the final product.

The gradation of the combined aggregates is an elementary, but important problem in the manufacture of concrete. The treatment of this one problem under the accepted methods of the design and control of concrete mixtures is a fair indication of the brand of intelligence used in that focal point of all of the knowledge we have of concrete. Mix design is not a matter of so many pounds of this and so many pounds of that; it is the considered application of knowledge of concrete and its ingredients to the requirements of the product.

Mr. Scholer discusses density, bleeding and segregation. I am inclined to disagree with him to some extent but neither he nor I will be able to obtain the desired results by accepting what Mother Nature provides and letting it go at that. Cements differ and aggregates differ. Many cements produce results that are desirable in some cases

and undesirable in others. Certain objectionable characteristics of cement and aggregate often can be corrected by proper mix design, yet a specified provision for such a correction is very rare indeed. It is much more common to find an engineer who enjoys a well deserved reputation for competency as a structural designer specifying a 1:2:4 concrete made from "good quality materials."

This may sound like a rather sweeping condemnation of engineers who specialize in structural design and specifications but I feel that it is not unwarranted. These conditions exist and little has been done to make any change. National societies intimately concerned with concrete continue to place all emphasis on materials. The need for uniformity and control of the characteristics of concrete has been relegated to second place. The attitude seems to be that the characteristics of concrete can be controlled only insofar as it is possible to control the primary materials.

Testing laboratories and organizations which specialize in the control of concrete construction maintain or should maintain at least one Engineer who is thoroughly familiar with the characteristics of concrete and with concrete construction. Such organizations are quite competent to work with the Designing Engineer in the preparation of specifications and to supervise the production of a concrete of predetermined characteristics. The manufacture of concrete is as much a specialized field as the manufacture of steel and when Designing Engineers recognize that fact we can obtain a product that is as dependable as any other good building material.

All of this is rather far afield from Mr. Scholer's subjects of discussion but each one of the phenomena mentioned by him is intimately involved in the problem of mix design. A gasoline engine, no matter how beautifully designed, will not run without a carburetor.

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Cracking in Concrete Due to Expansive Reaction Between Aggregate and High-Alkali Cement as Evidenced in Parker Dam*

By H. S. Meissner†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Excessive expansion has been recognized for some time as causing that type of concrete distress manifested by a characteristic random-pattern cracking. Usually catalogued under the generic term "unsoundness," it has been ascribed variously, sometimes specifically, to unsound aggregate or unsound cement, the latter further subdivided into high-free-lime and high-magnesia cements. In many cases failures could be but vaguely explained. However, recent disclosures point to an additional new explanation in the chemical reaction between high-alkali cement and the mineral constituents of certain aggregates. It is possible that many failures, which have been incorrectly interpreted, may ultimately be clarified when analyzed for this suspected action.

Following this discovery, additional examples of the cement-aggregate reaction have appeared, and it is now found to embrace such aggregate types as shale, siliceous limestone, andesite, granite, granite gneiss, rhyolite, felsite, chert, and phyllite, all found in many different locations. This gives rise to an uncomfortable suspicion that there are many mineral forms in aggregates capable of engaging in this destructive action and that the possibilities for encountering it are widespread. Next, to discover a remedy, it is incumbent upon concrete technicians to understand this disease.

The paper describes a chain of circumstantial evidence which connects one case of deteriorating concrete with this type of action. The cement used was known to have contained large amounts of soda and potash. After considerable research it was discovered that the natural sand and gravel contained small amounts of andesite and rhyolite,

*Received by the Institute March 11, 1941. Highlights of this subject were presented very much in brief by L. H. Tuthill for R. F. Blanks and H. S. Meissner, all of Bureau of Reclamation, at the Institute's Research Session, 37th Annual Convention, Washington, Feb. 18-20, 1941.
†Engineer, U. S. Bureau of Reclamation, Denver, Colorado.

which were reacting in the concrete with such cement. Part of the evidence was a gel substance, a by-product of the reaction, identified as sodium silicate.

By common conception, concrete is a mixture of inert fine and coarse aggregate bound together by a hardened paste of cement and water. The cement paste has been considered the only active component, whose activity declines as hydration of the cement proceeds. Evidence now appears, however, that cement engages in a pernicious form of activity with some aggregates, in addition to performing its normal function—thus also amending the popular notion that aggregates are inert.

Deterioration of concrete, as a result of chemical reaction between the alkalies in cement and mineral constituents in certain aggregates, was first observed and correctly diagnosed by Thomas E. Stanton.‡ From circumstantial evidence, he first deduced that such reaction was responsible for the defective condition of the concrete, and he was able to duplicate, in the laboratory, the expansive action between cement and aggregate which produced defective concrete. These findings were undoubtedly considered interesting by many readers but, at the same time, were regarded as representing an exceptional and unusual combination of circumstances. However, the possibilities that this condition is widespread, and that there are abundant types of aggregates, carrying minerals susceptible to attack by the alkalies in cement, are becoming apparent. Many cases of expansive disintegration, heretofore frequently ascribed to unsoundness, presumably due to free lime and magnesia, may instead have their explanation in this type of destructive reaction. The following account describes the deterioration of concrete on a large and recent structure and traces its origin to this reaction between aggregate and high-alkali cement.

Parker Dam (Fig. 1) is a concrete arch structure spanning the Colorado River 155 miles below Boulder Dam. It was completed in 1938, and, two years thereafter, the few existing cracks in the concrete were discovered to be enlarging and numerous new cracks were observed to appear suddenly and to extend themselves rapidly. Fig. 2 and 3 are illustrative of these cracks, which occurred in a random pattern over all exposed portions of the structure. The large crack, shown in Fig. 4, on the downstream edge of one of the trash-rack buttresses (seen to the left of the dam in Fig. 1) was found to extend the full height of this member. Similar wide cracks were found on the other buttresses. Random cracking is to be seen on

‡Influence of Cement and Aggregate on Concrete Expansion, by T. E. Stanton, *Engineering News Record*, Feb. 1, 1940.

FIG. 1—PARKER DAM IS THE WORLD'S DEEPEST, ITS FOUNDATION BEING 235 FEET BELOW RIVER BED, WHILE ONLY 85 FEET EXTEND ABOVE

almost all concrete surfaces, including the parapet walls, roadway, gate superstructure, piers between the spillways, and on the mass concrete buttress blocks. The cracks are less pronounced at lower elevations, and, near the water line, they either disappear or are of hairline order.

From the first critical survey covering the extent and nature of the cracks, the conclusion was reached that cracking was caused by some condition of unsoundness, coupled with the extreme conditions of drying in an arid climate. The visual evidence, as portrayed by the appearance of one of the newel posts at the ends of the parapet walls shown in Fig. 5, indicated that internal expansion of the concrete was aggravating the surface cracking. To determine the nature and amount of the volume changes occurring in the concrete, invar steel rods with attached dial gages were mounted on the concrete, some of which assemblies are to be noted in the accompanying illustrations. The measurements secured from these confirmed the first impression of expansive movement.

MEASUREMENTS ON THE CONCRETE

To obtain more factual data covering volume changes in the mass, three 3-in. diameter holes were drilled vertically, adjacent to each

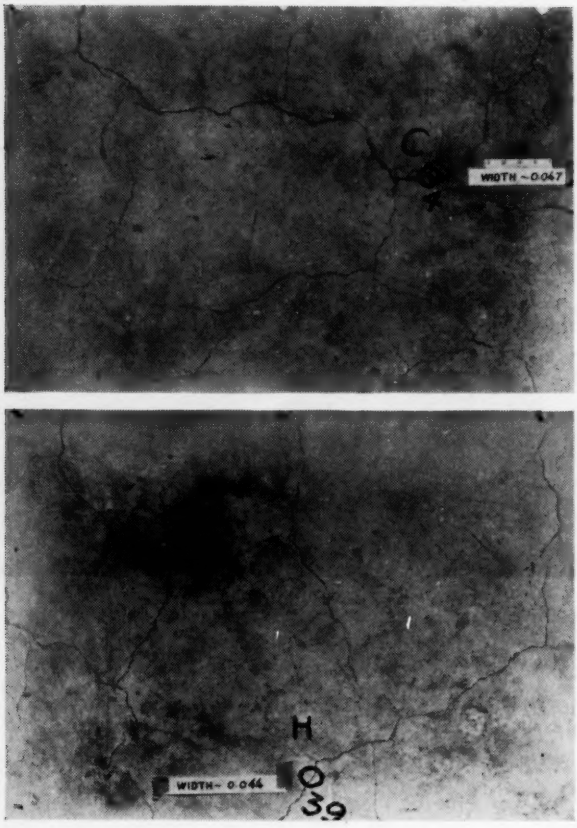
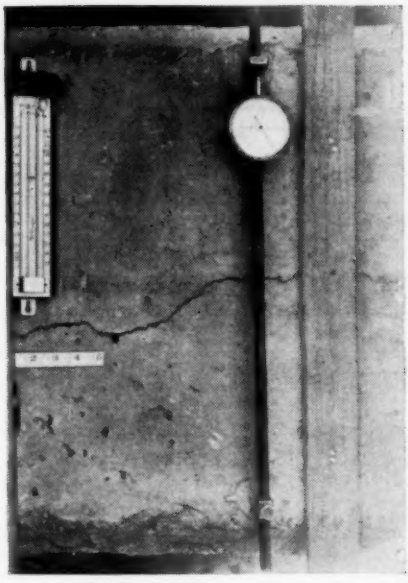


FIG. 2 AND 3—TYPICAL CRACKING IN CONCRETE, RESULTING FROM EXPANSIVE REACTION BETWEEN AGGREGATE AND CEMENT OF HIGH ALKALI CONTENT

other, on the roadway of the dam, into the concrete of one of the abutment sections, to depths of 2½, 5, and 10 ft. Invar steel rods with dial gages mounted on their tops were grouted into the bottom of these holes to register the over-all volume change in the concrete for these several distances from the surface. The holes were loosely packed with insulating material, to prevent air circulation and drying of the concrete, and resistance thermometers were installed to record concrete temperatures at several points. Fig. 6 shows the movements recorded by these meters for the first 2½ ft. of depth, the next 2½ ft., and the lowest 5 ft. These were computed by correcting the data for the 10-foot and 5-foot holes by that obtained on the hole next in depth.



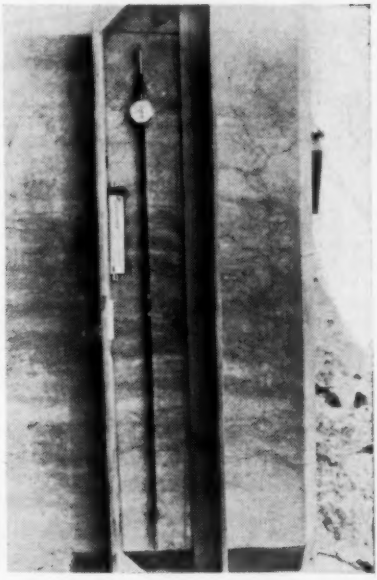


FIG. 4—CRACK, 0.11 IN. WIDE ON FACE OF TRASHRACK BUTTRESS

FIG. 5—CRACKS IN NEWEL POST CONVEY THE IMPRESSION THAT THE INTERIOR MASS OF CONCRETE IS ENLARGING AND CRACKING THE OUTER SHELL OF DRY CONCRETE WHICH DOES NOT PARTICIPATE IN THE MOVEMENT

The concrete is observed to be shrinking in the first 2½ ft. near the surface, from 2½ to 5 ft. below the surface it is expanding slightly, and from 5 ft. to 10 ft. below the surface it is expanding considerably. The picture thus presented is that of a large expanding interior mass of concrete whose relatively thin outer shell is contracting. It seems inevitable, therefore, that cracks should sooner or later form in this outer contracting shell. Its thickness is not accurately recorded by the data shown, but, from the depths of cracks observed on concrete cores, which will be mentioned later, it may be considered to be less than 12 in. Declining temperature seems to have counteracted expansion to some extent in that portion of the concrete from 2½ to 5 ft. deep. In the lower 5 ft., however, expansion continued uninterrupted at an astonishing rate. If these measurements are correctly representative of the volume change within the concrete, they register a movement within two and one-half months equivalent to that produced by an 80F. rise in temperature.

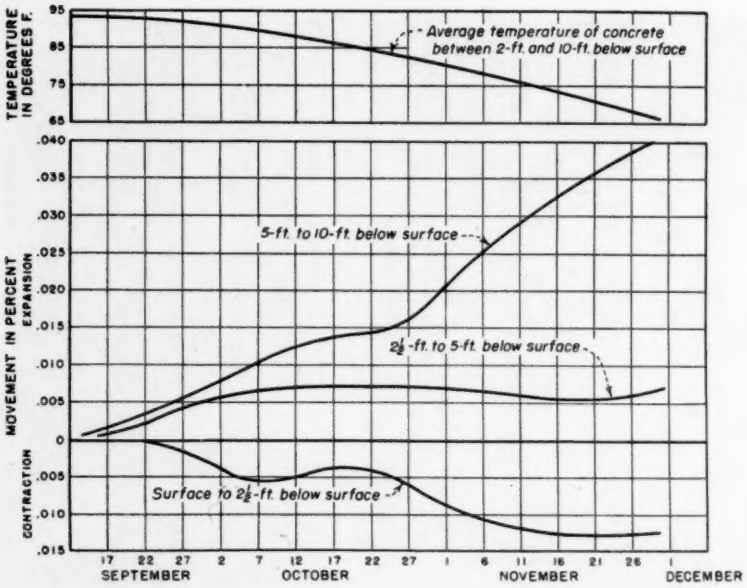


FIG. 6—INTERNAL VOLUME CHANGE IN ABUTMENT CONCRETE

SODIUM-SILICATE GEL

In a further effort to determine the causes for these conditions, a large number of cores, up to 10 in. in diameter, was extracted by diamond drill from various portions of the concrete. These cores were packed in damp sawdust and shipped to the laboratory, where, on unpacking, a white gel-like substance was found to be exuding from them and accumulating in several places on their surfaces (Fig. 7). Another type of excretion, seen in Fig. 8, developed as a clear, viscous liquid bead, which later hardened. Analyses of both of these deposits indicate them to be essentially sodium silicate, as suggested by their appearance.

Several of the cores of this defective concrete were broken up, and it was observed that many of the air void spaces were completely or partially filled with a mushy, clear to opaque, white gel (Fig. 9). On probing with a needle, this felt sticky and would dry tenaciously to the needle. Its appearance, like that of the exuding material, again suggested sodium silicate, which was confirmed by chemical analysis. The gel became hard on drying, accompanied by large shrinkage, and became carbonated.

Inasmuch as the determinations were made on very small samples, the results of chemical analyses on these substances, shown in Table 1, are subject to more than the usual analytical errors.

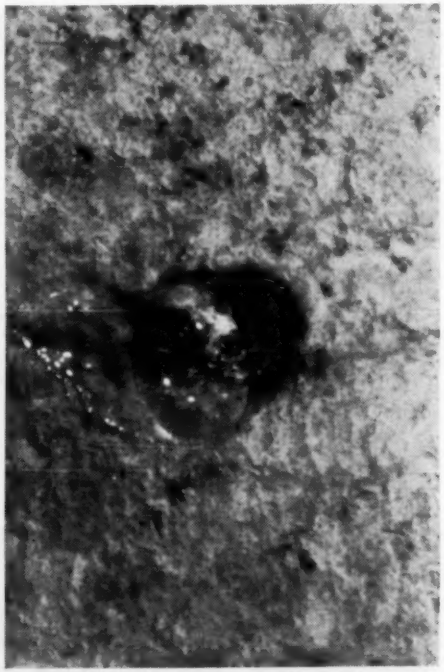


FIG. 7 (LEFT)—EXUDING DEPOSIT ON SURFACE OF CONCRETE CORE, WHICH IS FOUND BY ANALYSIS TO BE COMPOSED ENTIRELY OF SILICA AND THE ALKALIES, SODA AND POTASH

FIG. 8 (RIGHT)—BEAD DEPOSIT ON SURFACE OF CONCRETE CORE, WHICH IS INDICATED BY ANALYSIS TO BE ESSENTIALLY SODIUM SILICATE

FIG. 9 (LEFT)—BROKEN SECTION OF CONCRETE SHOWING GEL DEPOSIT FOUND WITHIN VOID SPACE IN CONCRETE CORE

FIG. 10 (RIGHT)—ANDESITE PARTICLE CONTAINING REACTING ZEOLITE

TABLE 1—ANALYSES OF DEPOSITS

	SiO_2	R_2O_3 + Insol.	CaO	MgO	SO_3	Na_2O	K_2O	Loss on Ignition
Beads	51.2%	....	....	....	0.0%	16.8%	*	....
Material from side of core	81.9	0.7%	1.1%	0.0%	....	4.0	2.0%	10.5%
Material from interior of core	53.4	1.5	4.7	0.0	0.0	12.7	4.5	20.5

*Included with Na_2O.

From an examination of these analyses, it appears that the beads and the material found in the interior of the core are similar in composition. The ooze taken from the outside of the cores is materially richer in silica and lower in alkalies and was different in physical appearance from the material within the core, giving the impression that hydrous silica had been precipitated from the viscous mass.

Since the origin of the gel was considered of primary importance, the concrete was closely examined with this in mind. It appeared at first that the gel was affiliated with a red zeolitic tuff aggregate and originated by base exchange with the zeolite (Fig. 10). Later it was found on the surface of a red nonzeolitic rock. This rock was literally filled with cavities and solidified veins of chalcedony, opal, and quartz. The vein of opaline silica was hard, sound, and unaltered in the interior of the aggregate particle, but, on the outer portion of the rock, it was relatively soft and could be probed with a needle. When first exposed in the freshly broken concrete, this piece of aggregate was bathed with a syrupy, viscous liquid which later dried to a hard glaze. Likewise, the soft portion of the opal vein hardened on drying. Analyses of the altered and unaltered portions of this material show definite reduction in the silica content with comparative increase in other components, especially soda and potassa. The gel has been found associated generally, however, with an andesitic type of aggregate (Fig. 11) which carries no zeolites. This is a very hard, igneous rock that would be judged sound from appearance alone and which, for the majority, is unaffected in the concrete; yet many of such pieces of aggregate are found in a softened and altered state within the concrete. Very frequently their surfaces show a zone of alteration and softening while their interiors remain intact. Fig. 12 presents this condition in a broken particle of such aggregate, showing a dark band at the contact between rock and cement paste.

A 6-in. cobble, with a completely etched and altered surface, was discovered in one of the cores broken for compressive-strength test. This rock was a hard, rhyolite porphyry with phenocrysts of feldspars, quartz, and mica, with a minutely crystallized matrix. The matrix

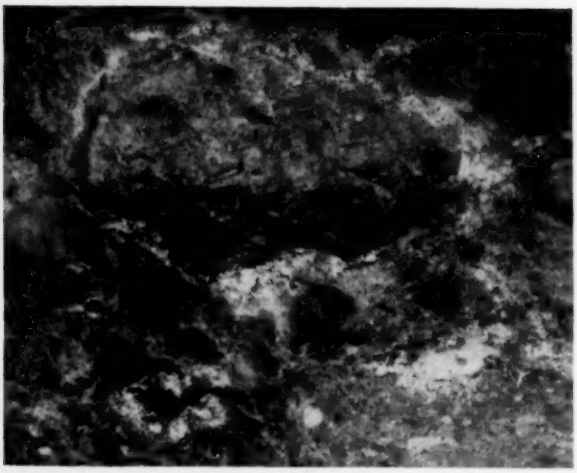


FIG. 11—ANDESITE AGGREGATE SURROUNDED BY EXUDING GEL

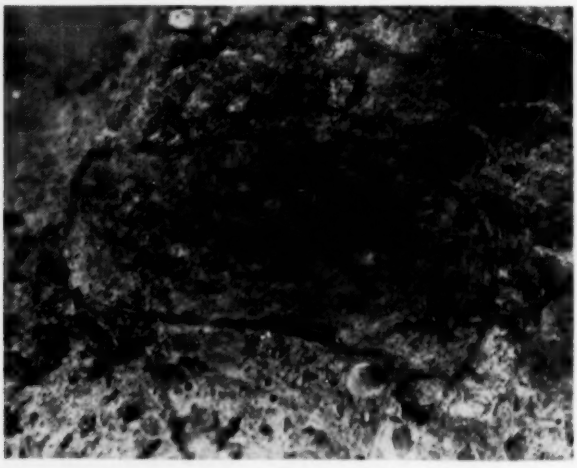


FIG. 12—BROKEN ANDESITE AGGREGATE PARTICLE WITHIN CONCRETE, SHOWING A DARK BAND OR ZONE OF REACTION ON ITS PERIPHERY

on the surface of this rock, to a depth of at least ⅛ in., had been so completely attacked that portions of it remained in the mortar socket when the rock was removed from the surrounding concrete. There was a colorless, thin, viscous film underneath this layer adjacent to the rock, which hardened to a glaze on drying. The interior of the rock itself was extremely hard, sound, and fresh. It would, of course,

have been possible for the surface of this rock to have become altered in the field, but this type of alteration would result in the oxidation of any iron-bearing minerals with resulting discoloration, and other typical changes. Such alteration was not evident in the decomposed layer. Instead, there was evidence indicating that the rock was chemically attacked while in the concrete. This was supported by the fresh appearance and angular structure of the feldspar and mica phenocrysts remaining in the altered layer. Any natural alteration of this type of feldspathic rock would occur first in the feldspars, and such alteration could be detected at once by the appearance of these crystals. This, in itself, was conclusive that dissolution of the matrix of this rock had taken place only during the time the rock was surrounded by cement and was further confirmed by the presence of gel found in the voids adjacent to this altered specimen. The chemical analyses of this rock and its outer shell are given in the Table 2. There is little difference between the compositions of these two materials although the silica is slightly lower and the alkalies slightly higher in the altered portion, but not enough so to be significant. A test for acid-soluble alkali showed this to be much greater in the shell portion of the rock, indicating that definite alteration has occurred but that the products of such reaction are only partially removed.

TABLE 2—ANALYSES OF ROCKS

	A	B	C	D	E
SiO_2	58.0%	83.0%	69.6%	70.2%	68.2%
Al_2O_3	16.7	6.6	5.1	15.1	13.8
Fe_2O_3	6.1	2.6	5.5	2.6	3.7
CaO	6.6	1.2	2.7	2.0	2.6
MgO	3.5	0.0	0.0	0.64	0.64
Na_2O	2.2	2.7	7.8	3.4	3.8
K_2O	4.0	2.1	4.1	3.8	4.8
Loss (950° C.)	2.7	2.0	6.7	1.0	1.2

A—Andesite rock.
B—Opal vein in fractured andesite.
C—Disintegrated sample of "B".
D—Unaltered rhyolite.
E—Altered shell of rhyolite.

In addition to the gel deposits and evidence of alteration in aggregate particles, the paste structure revealed in a freshly broken surface of Parker Dam concrete presented a hackly and granular appearance and had a lifeless cast that distinguished it from other more healthy looking concrete. The cores, being broken apart for examination, also emitted a dull thud when struck with a hammer, in contrast to the ring with which concrete usually resounds.

HIGH-ALKALI CEMENT

As the foregoing information accumulated, attention was directed toward the cement used in building Parker Dam. This was a low-heat

type secured from four mills, used as received, without any blending other than that secured by handling simultaneous shipments through the storage silo and mixing-plant bin at the project. One brand of cement was used entirely in the last stages of the work, particularly in the concrete above the spillway crest and in certain of the trash-rack buttresses, all of which is affected by cracking. The other brands of cement cannot be separately identified in the concrete. Some slight difference was observed in the trashrack buttresses between concrete in which all four brands of cement were used and that in which one brand alone (containing the highest alkali) was used, the cracking being more pronounced in the latter. From their analyses, it will be noted that all brands, except one, were high in alkali, the highest having 1.42 per cent.

MORTAR EXPANSION TESTS

The suspicious connection between alkalies found to be present in the gel deposits and the high content in the cements immediately suggested reaction between such cement and the aggregate, as previously announced by Thomas E. Stanton.* Following his procedure for determining such reaction, 1:3 mortar bars were fabricated with one of the Parker Dam cements and the sand used in that structure. These were sealed in metal cans in the presence of, but not in contact with, water and stored at 100 F. Two sands were used at Parker Dam, a natural sand in which the andesite particles are found (although only three per cent by weight is observed), and a blow sand, composed essentially of clean quartz grains, which was blended in the concrete mix to improve the aggregate grading.

These two sands, together with a sand similar to the Parker sand obtained from a nearby deposit (Graham Hills) and Platte River sand, for comparative purposes, were used in making mortars in combination with cement containing 1.13 per cent total alkalies. Another group of mortar bars was cast with these same materials, but containing, in addition, one percent sodium hydroxide, by weight of the cement. These bars were periodically measured, by means of metal inserts in their ends, to record the expansions plotted in Fig. 13. After curing in the sealed state for several weeks, those specimens made with Parker Dam sand were observed to be speckled with oozing material, resembling somewhat that seen upon the concrete cores. This was more pronounced on the bars containing the added sodium hydroxide, and close examination of these bars revealed fine hairline cracks on

*Expansion of Concrete Through Reaction Between Cement and Aggregate, by T. E. Stanton, *Proceedings*, A. S. C. E., Dec. 1940, Vol. 66, p. 1781.

TABLE 3—ANALYSES OF FOUR BRANDS OF LOW-HEAT CEMENTS USED IN PARKER DAM

	PV	PC	PM	PR
SiO_2	26.60	22.71	22.72	22.11
Al_2O_3	3.14	5.17	5.71	5.27
Fe_2O_3	1.53	5.07	5.74	5.45
CaO	63.71	59.89	59.84	58.97
MgO	2.12	3.06	1.16	4.27
SO_3	1.76	2.04	2.07	1.68
Loss	1.12	1.43	1.68	1.55
Na_2O	0.13	0.69	0.81	0.82
K_2O	0.42	0.56	0.61	0.31
Na_2O — K_2O	0.55	1.25	1.42	1.13
Insol.	0.12	0.06	0.13	0.08
Free lime	0.20	0.16	0.35	0.18
C_3S	28.9	23.6	17.9	23.6
C_2S	54.5	47.1	51.6	45.6
C_3A	5.7	5.1	5.4	4.7
C_4AF	4.6	15.4	17.4	16.6
$CaSO_4$	3.0	3.5	3.5	2.9
Merriman Tests				
Alkalinity	2.2	4.4	4.5	3.7
Free alkali	3.3	8.2	4.9	3.1
Sugar cloud	2.5	2.4	2.3	2.0
Sugar clear	2.5	2.7	2.3	2.0

their surfaces. It will be observed that, after 20 weeks storage, the mortar bars to which one per cent NaOH was added expanded 0.16 per cent. The specimens containing Graham Hills sand and added alkali also showed considerable expansion. Neither the Platte River sand nor the blending sand seemed to have been materially affected by added NaOH.

Additional tests with Parker sand and a cement of low alkali content show less expansion than observed in experiments with a high-alkali cement. The reaction between aggregate (or certain constituents of the Parker aggregate) and high-alkali cement, which was suspected from measurements on the dam and from the evidence presented by the concrete cores, seems, therefore, to be substantially confirmed.

FIVE-YEAR-OLD LABORATORY CONCRETE

Additional corroborative data were obtained from an examination of a few laboratory specimens, made during early studies of Parker Dam concrete, prior to construction of the dam, and which had been retained to secure long-time compressive-strength values. These were mass-cured specimens made with a modified-type cement containing 0.91 per cent alkalies, sealed in metal containers, and, at the time cracking in the dam became acute, had reached an age of five years. On opening the containers, it was found that all cylinders showed activated areas, from ¼ to 1¼ in. in diameter, on their surfaces. One had an appreciably thick layer of white consolidated material on the top which would not effervesce with acid, but which reacted as hydroxide with phenolphthalein. This material, although rather hard, was not cracked when the can was opened but immediately

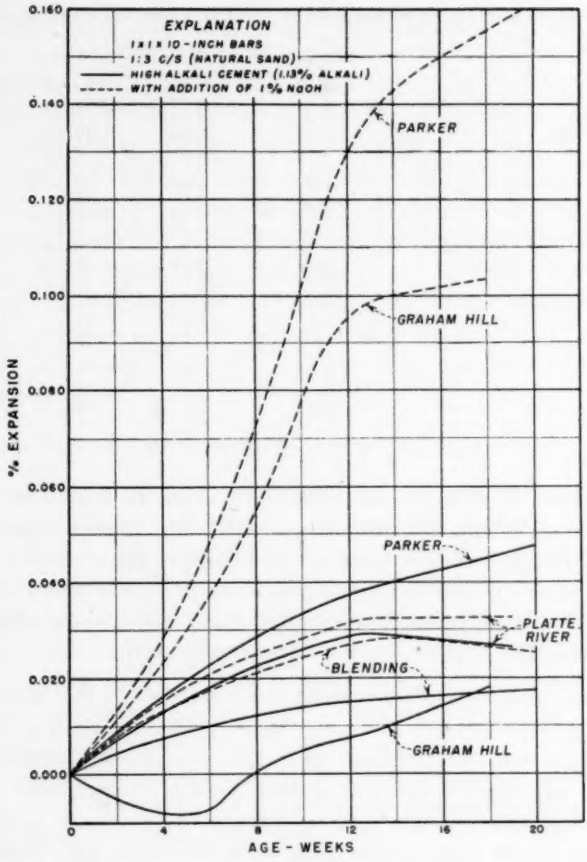


FIG. 13—EXPANSION OF MORTAR BARS AT 100F. SEALED IN PRESENCE OF MOISTURE

began to shrink and form scales on exposure to air. It was very similar to some of the dried exuding deposit noted on the cores from Parker Dam.

When a straightedge was laid over the activated areas, many of them were observed to protrude above the surrounding area. Some of the activated areas were opened up with a knife, and, within a few of these, the red andesitic aggregate particles were visible. A popped-out fragment showed a clear gel on the surface of the broken rock. This gel was watery and viscous when the concrete flake was first lifted from the specimen but rapidly dried to a glaze. Companion specimens, which were fog-cured for the same period, did not show any

of the unusual characteristics mentioned above for the sealed, adiabatic, mass-cured specimens.

STRENGTH OF CONCRETE CORES

In addition to the concrete cores previously mentioned, another group of 6-in. diameter cores was taken from the dam during the construction period. These were cut from the dam when the concrete was six months old and stored in the open laboratory until they had reached an age of 2¼ years. They were tested for compressive strength and elastic modulus after being soaked in water for several days. In Table 4 the results of these tests are shown compared to similar values obtained on the more recent cores. The latter were likewise tested in a saturated state at an age of three years, for which period the concrete was, of course, curing within the structure. For comparative purposes, the 28-day control strengths for this concrete are presented in the table as well as one-year values on mass-cured laboratory concrete made with a modified-type cement.

TABLE 4

	Age	Compressive Strength p.s.i.	Modulus of Elasticity
Early cores	2¼ years	4,180	3,800,000
Recent cores	3 years	3,480	2,200,000
Control Cylinders	28 days	4,120	.....
Laboratory concrete 6- by 12-inch cylinders	1 year	4,620	4,200,000
Laboratory concrete 18- by 36-inch cylinders	3 months	3,980	4,800,000

It will be noted that the cores recently taken from the structure, although older and having received more favorable curing within the body of the parent concrete, have lower strengths than the earlier cores and lower strengths than less mature laboratory concrete. What is more significant, the modulus of elasticity (which is coming to be recognized as an important index of the relative durability and condition of concrete) of those cores representing the present concrete within the dam is of a very considerably lower order. For individual specimens, this was found to be as little as 1,000,000, indicating definitely that retrogression and deterioration are occurring in Parker Dam concrete.

DEPTH OF CRACKS

The slight depth to which even the largest cracks were found to extend was another surprise revealed by the cores. The crack shown in Fig. 14 was found to be 0.11 in. wide on the surface and was expected to penetrate some distance into the concrete. Actually, the crack was

FIG. 14—CORES REVEAL THAT EVEN THE WIDEST CRACKS ARE OF SLIGHT DEPTH

discovered to taper rapidly and to have a depth of only six to eight inches. It may be detected by the aid of magnification for a few additional inches, but certainly no farther. Several other cores at different points proved the cracks to be even less extensive in depth.

That the surface cracks are associated to some extent with drying shrinkage at the surface is indicated by the observed depth of cracks and the computed depth to which water is lost from the concrete by evaporation. Following the methods for such computations given by Carlson,* who has shown that the loss of evaporable water from concrete, continuously exposed to a given uniform relative humidity, can be calculated in the same manner as loss of heat, it is estimated that loss of more than 50 per cent of the evaporable water in two years is limited to a depth of 3½ to 11 in.

T. E. Stanton, whose work has previously been mentioned, found that the delayed expansion, accompanying reactions between aggregate and high-alkali cement, is promoted by and continues only in

*Carlson, Roy W., Drying Shrinkage of Large Concrete Members, JOURNAL, Amer. Concrete Inst. Jan.-Feb. 1937, pp. 327-336.

the presence of moisture. This has been corroborated by some tests of this present work. Since the depth to which the cracks extend is also the zone in which reaction has been inhibited by drying, the zone immediately below may be considered as that in which the action is being continued. Contraction, through drying, therefore, stops abruptly at the junction of these two fields. Widening of the crack at the surface is also encouraged by interior expansion, and its growth in depth proceeds no farther than the zone of drying since expansive effort is encountered immediately beyond that point. We may, therefore, have a condition which, though producing and enlarging cracks at the surface of the concrete, impedes progress of those cracks to any great depth. The patterns and nature of the cracks are exactly what may be expected to occur on a mass, whose volume is increasing, if its outer shell could not accompany this expansion and were, instead, shrinking. Near the water's edge, where cracking has not progressed, the surface concrete has not dried to the same extent as elsewhere and, instead of shrinking, has expanded in conformity with the interior concrete.

SPECULATIONS

Of the many and complex types of rocks noted in the Parker aggregate, only a few show perceptible activity in the samples of defective concrete. Some of the rocks that have shown activity within the concrete appear to have had their surfaces rendered vulnerable to attack by weathering. A number are porous, fragmental tuffs, hence have large surface area exposed to the cement paste and are capable of absorbing considerable water and soluble salts. Such rocks, and those that have been partially altered in the field by natural weathering agencies, are in such condition that corrosive solutions would greatly accelerate their decomposition. This decomposition would, of course, be a gradual process from the exterior of the rock toward the center and would diminish in speed as penetration is made and as the solutions tend to neutralize themselves (see Fig. 12).

The suspected chemical reaction between the aggregate and the alkali of the cement has not yet been absolutely demonstrated. However, investigators in different lines of research have shown that minerals, such as feldspars, are subject to attack by sodium. E. A. Stephenson* has shown the following reaction between a feldspar and sodium (as carbonate):

$$\underset{\text{(albite)}}{NaAlSi_3O_8} + H_2O + Na = \underset{\text{(analcite)}}{NaAlSi_2O_6 \cdot H_2O} + SiO_2 + Na$$

*Journal of Geology, Vol. 24, 1916, p. 180.

In the reaction products, he obtained acicular analcite crystals and a heavy gelatinous precipitate consisting of silica and water. He demonstrated by test the possibility of definite attack and dissolution of the feldspars with loss of one molecule of silica from each molecule of feldspar at no expense to the caustic alkali which remained free to continue the destruction. Silica, to the amount of 5.1 per cent and 8.2 per cent, has been dissolved from Parker andesite and rhyolite crushed to No. 14 sieve size grains after 25 days exposure to 10 per cent sodium-hydroxide solution. Silica and sodium have been identified in the exuding gel from the Parker Dam cores. Therefore, it would appear that the reaction between cement and certain aggregates in Parker Dam concrete results in end products that are similar to those obtained above by Mr. Stephenson.

The most important type of activity noted is that in which a shell of altered material is formed on the aggregate, as may be seen in Fig. 12. Most of the white gel was found in natural openings of the concrete that may or may not have had access to the rocks indicated, but, in every case, there was an andesite close at hand. There also appeared evidence indicating movement of the gel along small lateral openings in the concrete with larger openings acting as collective areas for the material.

There was no visible indication that the free gel itself was causing the expansive pressure, as in most cases it did not even fill the voids in which it was located. However, there are other factors which may indicate swelling, such as, for instance, the formation of gel between aggregate and mortar. This type of deposition seemed to have caused the gel to be forced out onto the surface of the core and around the indicated piece of aggregate. Also, it has been shown that, when a silica gel dries, it has tremendous shrinkage, hence was expanded when hydrated. While there is no proof that this type of expansion is accompanied by pressure, it may logically be suspected. The gel may more easily be considered the byproduct of a more complex reaction, the hydrous silicate of which, if it has an increased volume that cannot be locally accommodated, may cause pressure as it forms.

RETROSPECTION

Parker Dam concrete was manufactured with all the care and precautions considered necessary and desirable for the production of a high-quality product. The source of aggregate was selected after much exploratory work, considering several other possible deposits, and after considerable preliminary laboratory study. From early large-scale laboratory mixes, it was concluded that satisfactory con-

crete could be produced from these materials. The aggregate was subjected to a petrographic examination and to practically all known tests to determine its suitability, with the resulting conclusion that it was very acceptable material.

The operations of the deposit and aggregate plant were designed to improve its use as a concrete ingredient. Effort was made in the processing of sand and gravel to provide uniformity by classifying the sand into two sizes and by separating the coarse aggregate into four sizes. The aggregate was washed to remove dirt and silt. Batching and mixing were performed in a modern plant which included all the latest refinements known to secure accurate proportions and control of the concrete consistency. Placing was carefully supervised and the concrete was thoroughly cured by continuous application of water for a minimum period of two weeks, except for some portions of the structure where curing compounds were applied in three coats.

In spite of all efforts taken to produce high-quality concrete and the observation of every precaution considered necessary to secure good work, the concrete in Parker Dam is now, 2½ years after its completion, found to be deteriorating. The possibility of reaction between aggregate and cement had, of course, not been revealed prior to this time. Even if this had been suggested, it is doubtful that those elements of the aggregate, now observed to be undergoing change in the concrete, would have been regarded with suspicion until after extensive tests. These experiences suggest the need of a new criterion for judging the suitability of aggregate and the necessity for being more critical of the cement with respect to its alkali content. The common conception of aggregate as an inert filler in concrete must be revised.

CONCLUSIONS

1. Cracking in the concrete of Parker Dam is general over all parts of the structure exposed to view. This is generally of a random pattern, which, from its character and appearance, suggests expansion within the concrete.

2. In the trashrack structure, where several brands of cement were used, the degree of cracking seems to vary somewhat with the cement used, being more pronounced where a cement known to have contained 1.42 per cent total alkali was used.

3. The four different brands of low-heat cement used in Parker Dam concrete are found to have contained 0.55, 1.13, 1.25, and 1.42 per cent total alkali. The latter cement was used to the greatest extent and where cracking has been observed.

4. The cracks, even those as wide as 0.11 inch, are found to extend only six to eight inches into the concrete.

5. Concrete cores taken from Parker Dam show inferior compressive strength and low modulus of elasticity.

6. Concrete cores taken from the dam and trashrack structure have an exuding deposit upon them and when broken open contain deposits of gel which are revealed by analyses to be composed mainly of silica, soda, and potassa. The surfaces of the cores as well as their interiors, when broken, show altered and defective pieces of andesitic aggregate.

7. There is good reason to believe that the exuding deposits have their origin in certain andesite aggregate particles whose decomposition results from contact with high-alkali cement.

8. Linear measurements, made with dial gages attached to invar rods placed in holes drilled in the mass concrete, show the concrete to be shrinking at the surface to a depth of 2½ ft.; from 2½ to 5 ft. below the surface, expansion is registered; and from 5 ft. to 10 ft. below the surface, a considerable progressive expansion is recorded. Within a 3-month period a unit expansion of 400 millionths has been observed in this latter zone.

9. Five-year-old laboratory specimens of concrete made with Parker Dam aggregate and a cement containing a total of 0.91 per cent alkali showed deposits on them and also revealed defective andesite aggregate in pop-outs on their surfaces.

10. Mortar bars of 1:3 mix made with Parker Dam sand and high-alkali cement, cured in sealed containers in the presence of moisture, have expanded 0.05 per cent in 16 weeks. Companion bars to which one per cent *NaOH* by weight of cement was added have expanded 0.11 per cent in this time. Mortars made with Platte River sand give negligible expansions in these same combinations. Lower expansions have been obtained with Parker Dam sand and cement of lower alkali content.

11. The cracking is due to the use of cement containing inordinately high amounts of alkali and to a chemical reaction between such cement and certain constituents of the aggregate.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the JOURNAL Supplement concluding this Proceedings volume

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by H. S. Meissner:

Cracking in Concrete Due to Expansive Reaction Between Aggregate and High-Alkali Cement as Evidenced in Parker Dam*

BY JULIAN HINDS AND LEWIS H. TUTHILL, HERBERT INSLEY AND AUTHOR

BY MESSRS. HINDS† AND TUTHILL‡

Mr. Meissner has ably described and discussed the evidence which indicates that the cracking at Parker Dam is the result of an unfavorable reaction between certain elements of the cement and of the aggregate. To this evidence there may be added certain significant facts concerning concrete structures located only a few miles from Parker Dam in the Colorado River aqueduct system of the Metropolitan Water District of Southern California.

The Gene Wash Dam and the Copper Basin Dam are two thin arch dams forming desilting reservoirs near the intake of the aqueduct. The Gene Wash Reservoir is filled by the Intake pumping plant from Lake Havasu formed by Parker Dam, and the Copper Basin Reservoir is filled by the Gene Pumping plant from the Gene Wash Reservoir.

The two dams were built at the same time Parker Dam was built, by the same contractor, using aggregates from the same deposit for all three structures. Low-heat cement for all three dams was supplied from the same mill under the same specifications. (Cement from other mills was used only in what is now the submerged portion of Parker Dam.) Because of their thinness and the use of low-heat cement which requires ample curing, these dams were cured with two coats

*ACI JOURNAL, Apr. 1941; *Proceedings* V. 37, p. 549.
†General Manager and Chief Engineer, Metropolitan Water District of Southern California.
‡Formerly, Concrete Engineer, Metropolitan Water District of Southern Calif.—now Bureau of Reclamation.

of a coal-tar cutback sealing compound followed by whitewash. This method was adopted in preference to a nominal water curing in order to prolong curing and forestall drying as long as possible. Nevertheless cracking of the same character as has appeared on the Parker Dam has shown up on both these dams although as yet not quite to the same degree. Drill cores show the same exudations of gelatinous material, the same formulation of reaction products in small voids in the aggregate and air holes in the concrete, the same peripheral alteration around certain pieces of andesite and rhyolite aggregate, and a similarly low modulus of elasticity and strength, as shown in Table A.

TABLE A—COMPARATIVE STRENGTH AND MODULUS OF ELASTICITY

	Parker Dam	Metropolitan Water Dist. Dams	Pump Houses
Compressive Strength—6x12-in. Specimens			
1938 28-day cylinders	4120	3450	3030
1940 drill cores	3480	3295	5775
Modulus of Elasticity—6x12-in. Drill Cores			
1938 test	3,800,000	.....	.....
1940 test	2,200,000	2,420,000	4,800,000

The pumping-plant structures were built about the same time using similar Colorado River system aggregates. Although these aggregates did not come from the same supply points, they were from terrace material deposited by the river apparently at about the same time a short distance downstream from the material used in Parker Dam and contained the same petrographic elements. (Steps are being taken to identify minute differences which may exist in the petrographic elements appearing to be similar in the usual routine classification of rocks but which may prove to be important to the potential reactivity of aggregate materials in concrete.) The cement was different from that in the dams, being a modified cement from another mill. The concrete received nominal water curing. In contrast to the dams, these pump houses show no deterioration, no signs of cracking, very little surface crazing, and drill cores show no evidence of cement-aggregate reaction, or reduction in modulus of elasticity and strength, as shown in Table A.

It was the practice of the District to store for reference a 4- or 5-pound sample of cement from each 1,500 bbl. of cement in each mill bin tested for the work. Records were kept of all places where any bin of cement was used, and of the bin from which cement was used in each part of the work. This is a practice which should be followed wherever possible in concrete work. The cost is negligible compared with the value of results which may be obtained from later special

tests of the cement in connection with investigations of the causes of subsequent performance of the concrete.

From samples of cement used in the dams and in the pump houses thus available for test, it was found that the cement from that in the dams contained 0.82 per cent Na_2O and 0.64 per cent K_2O, total 1.46 per cent, while the sample from the pump-house cement contained only 0.13 per cent Na_2O and 0.38 per cent K_2O, total 0.51 per cent.

In this case there is clear correlation between reacted aggregate, cracked concrete, and high-alkali cement on one hand and (as far as is known) similar but unreacted aggregate, sound concrete, and relatively low-alkali cement on the other.

BY MR. INSLEY*

In the section of this paper entitled, "Speculations," experimental work of E. A. Stephenson† is cited as evidence of the attack of "alkali" on feldspars. In his experiments Stephenson used a number of different solutions, including water, fluorides, borates, sulphides, aluminates, and sodium carbonate and bicarbonate. Because only the sodium carbonate solution is of interest in this discussion, Table B giving the complete results with this solution is copied without change from Stephenson's paper (p. 190). The minerals studied were adularia and microcline, the potassium feldspars, and albite, the sodium feldspar, one gram of mineral crushed to pass a No. 100 sieve being used in each experiment. For temperatures above 100 C. and pressures above 1 atmosphere, the reacting materials were inclosed in copper bombs and were heated in thermostatically controlled ovens.

Stephenson's work shows no evidence of reaction at 15 C for 3 years or at 100 C for 90 days. Only at a temperature of 185 C and a pressure of 11 atmospheres did he find any evidence of reaction products. Such conditions are so far beyond the range of anything encountered at any time since the production of the concrete in Parker Dam that these experiments do not appear pertinent.

*National Bureau of Standards, Washington, D. C.—Published by permission of the Director, National Bureau of Standards, Washington, D. C.

†*J. Geology*, V. 24, pp. 180:199 (1916).

TABLE B

No.	Mineral	Time	Solution	Concentration	Vol. Sol.	Vol. Tube	Temp.	Press. Atm.	Results
5	Andularia	3 yrs. 1 mo.	Na_2CO_3	N/2	1,000 cc	..	15°C. ca.	1	No change
6	"	90 days	"	"	300	..	100°C.	1	"
7	"	82	"	"	50	80 cc	183°C.	11	Analcite
8	"	18	"	"	40	85	233°C.	30	"
9	Microcline	18	"	"	40	75	233°C.	30	"
10	Albite	18	"	"	40	85	233°C.	30	Tube burst

Of possibly more significance than the presence of feldspar in some of the reactive aggregates is the fact that siliceous glass, of either the opaline or volcanic type, has been found in all the aggregates that were suspected of reacting with the alkalies either in the Parker Dam or in the cases investigated by Stanton.*

The difficulty of distinguishing the contribution of single constituents to the reaction with alkalies in aggregate pieces which are, almost without exception, mixtures of many constituents, suggests a method of attacking the problem by the use of laboratory concrete specimens containing aggregate made up of single mineral constituents. Feldspars of several kinds, natural glasses, silica minerals (including the several varieties), magnesite, dolomite, etc., can be obtained from essentially monomineralic rocks or by separation from heterogeneous rocks. Other valuable information could be obtained by a study of the reaction between such homogeneous aggregates and alkaline solutions.

AUTHOR'S CLOSURE

Since publication of this paper, interest in the question of cement-aggregate reaction has grown. Evidence that this action may account for similar trouble experienced in other deteriorating concrete has been given in two subsequent papers by Bailey Tremper† and H. A. Kammer and R. W. Carlson‡, appearing in the June 1941 issue of the JOURNAL. Discussion of Mr. Stanton's article in Proceedings of the American Society of Civil Engineers, previously referred to, will also be found interesting to the reader. In an effort to throw light on this perplexing problem, the Bureau of Reclamation has initiated a comprehensive program of research, and to secure the cooperation of cement manufacturers, a conference was held in the early part of this year, widely attended by manufacturers operating in the Bureau's territory. The cement producers are assisting in this research through a technical committee and are supplying cements covering a wide range of alkali content.

Included in this program is a study of several cases of concrete disintegration, reported in widely scattered locations in the country, which have been suggested as appropriate clinical subjects. With these, in general, the deterioration has followed a pattern, associated with internal expansion manifesting itself in random cracked surfaces, accompanied frequently by exudations or having coatings of exuded

*T. E. Stanton, Proc. Am Soc. Civil Eng., V. 66, p. 1781 (Dec. 1940).

†Bailey Tremper, Evidence in Washington of Deterioration of Concrete through Reactions Between Aggregates and High-Alkali Cements, ACI JOURNAL, June 1941, *Proceedings* V. 37, p. 673.

‡H. A. Kammer and R. W. Carlson, Investigation of Causes of Delayed Expansion of Concrete in Buck Hydroelectric Plant, ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 665.

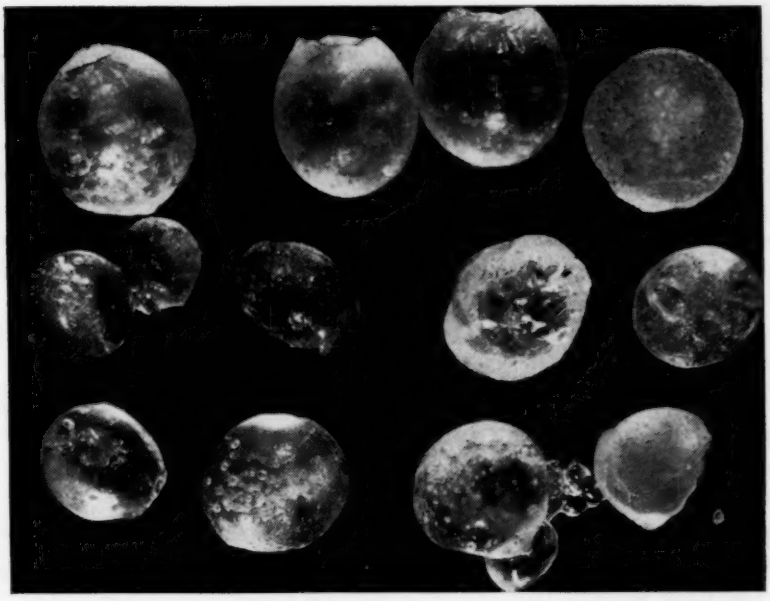

FIG. A—EXUDED BEADS GATHERED FROM THE PARAPET WALL AT PARKER DAM (ABOUT 3¾×)

material. While the cement-aggregate reaction is only considered a possibility in these cases, a study of all of them may provide from some the information needed to help solve this problem.

Foremost in this work is the petrographic examination of defective concrete. Prospects are excellent that a critical study of thin and polished sections of the concrete will reveal much. Emphasis is to be placed on the structure of the concrete matrix, the contacts of matrix and aggregate and the examples of disintegrating aggregate which, as in Parker Dam concrete, appear to be the source of sodium-silicate gel. By microchemical analyses and the determination of optical properties, it is hoped to establish the identity of the reaction products seen in the rims of reacted aggregate particles, such as shown in Fig. 12. If reactive aggregates show response to small scale tests, the microscope may be found useful in detecting and avoiding them.

The discussion by Messrs. Hinds and Tuthill well supports the contention that the disruptive expansion occurring in Parker Dam is related to the aggregate-cement combination. Where the same materials were again used, identical symptoms have appeared. In contrast, a similar aggregate used with low-alkali cement in the pumping plant gave successful results.

The author is indebted to Mr. Insley for presenting the details of E. A. Stephenson's experiments but he cannot agree that this work is not pertinent. Anything to do with the alteration of aggregate minerals, even though it may only be remotely associated, is pertinent to this important problem. While the temperatures used in Stephenson's tests are above those met with in Parker Dam, the interior temperatures in this structure have been as high as 140 F. Also, since undue expansions have occurred, it may not be unreasonable to assume local pressures much greater than the 11 atmospheres found necessary to alter the feldspar. Reactions which require heat or pressure, or both, for acceleration are very often completed under normal conditions with the passage of time.

Tests on specimens containing homogeneous aggregates, composed uniformly of one mineral in each case as suggested by Mr. Insley, have been in progress. It may be interesting to observe, in view of the above discussion, that one of the largest expansions secured has been obtained with a high-alkali cement mortar containing a microcline-perthite feldspar.

The data secured from observations at Parker Dam since publication of the original article strengthen and support the conclusions reached therein. Fig. A shows a number of translucent beads which have since appeared on the surface of the concrete on top of the parapet wall, in some areas as many as 20 beads per square foot. Their composition is essentially sodium silicate, as shown in Table 1 for similar exuding material.

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Columns With High Yield Point Reinforcement Designed Under the A. C. I. Code*

By Thor Germundsson†

SYNOPSIS

The customary type of reinforced concrete column is not too satisfactory for large loads. An improvement is proposed by means of which columns may be made smaller and more economical. At the same time, their size may be kept the same throughout a tall building, 20 in. being considered the optimum size for all columns carrying up to 1,000,000 lb. This results in standardization of design, detailing and erection of floor construction on various levels. To accomplish that it is proposed to use 8 bars in each column, the maximum size of bars being 2 in. round and their yield point stress 75,000 p.s.i.

INTRODUCTION

The prevailing practice in column design is to use reinforcement of intermediate grade with yield point stress of 40,000 and working stress of 16,000 p.s.i. The footnote in Section 1103(a) of the A. C. I. Code, 1936, reads in part: "Nominal working stresses for reinforcement of higher yield point may be established at 40 per cent of the yield point stress, but not more than 30,000 p.s.i. . . . ". A stress of 30,000 p.s.i. is therefore allowed on bars with a yield point stress of 75,000 p.s.i.

It is the purpose to present and discuss the type of column that can be designed under the A. C. I. Code with a working stress of 30,000 p.s.i. The maximum load allowed on a 20-in. square column with 3,000-lb. concrete is 1024 kips. To carry the same load with inter-

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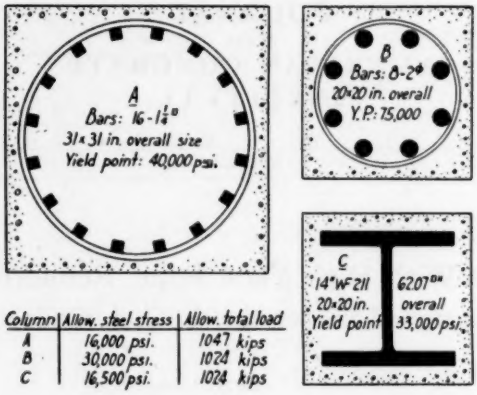
†Portland Cement Assn., Chicago.



Column	Allow. steel stress	Allow. total load
A	16,000 psi.	1047 kips
B	30,000 psi.	1024 kips
C	16,500 psi.	1024 kips

FIG. 1

mediate grade bars the column size must be increased to 31 by 31 in. The cost of the 20-in. column is one-third less than the cost of the 31-in. column. This is but one of the advantages of the smaller column.

DESCRIPTION OF COLUMNS

Fig. 1A shows a 31-in. column designed according to prevailing practice with intermediate grade reinforcement and a 3,000-lb. concrete. It can carry a load of 1047 kips and occupies 6.67 sq. ft. of floor area. The area of bars, 24.96 sq. in., is the maximum that can be used under spacing requirements for lapped splices, but the steel ratio is only 0.026 or less than one-third of the maximum ratio allowed.

Fig. 1B shows a 20-in. column with eight 2-in. rd. bars having a yield point stress of 75,000 p.s.i. It can carry a load of 1024 kips and occupies 2.78 sq. ft. of floor area. The area of bars, 25.13 sq. in., is practically the same as in the 31-in. column, but the steel ratio is increased to 0.063 for a 20-in. square section and to 0.08 for a 20-in. round section. Fig. 1C is a 14-in. H-column with 2-in. concrete protection and 20-in. over-all size.

COMPARISON OF COLUMNS

Table 1 contains a comparison of various quantities for columns marked *A* and *B* in Fig. 1. The 20-in. column is seen to be superior in every respect. Compared with the 31-in. column, the reductions in quantities are:

TABLE 1—COMPARISON OF COLUMNS DESIGNED UNDER A. C. I. CODE
1½-in. Protection, 3000-lb. Concrete, 12-ft. Story Height

		31-in. Col. (Fig. 1A)	20-in. Col. (Fig. 1B)
Yield point, bars, p.s.i.		40,000	75,000
Yield point, spirals, p.s.i.		40,000	60,000
Column Size, in.		31 x 31	20 x 20
Cross-section, area, sq. ft.		6.67	2.78
Load, kips	Concrete	648	270
	Bars	399	754
	Total	1047	1024
Bars	Number and size	16—1□	8—2ϕ
	Area, sq. in.	24.96	25.13
	Ratio	0.026	0.063
Spiral		⅝ ϕ—2¼″—28″	½ϕ—2¾″—17″
Cost	Concrete, cu. yd.	2.96 @ 10.00 = 29.60	1.23 @ 10.00 = 12.30
	Forms, sq. ft.	124 @ 0.25 = 31.00	80 @ 0.20 = 16.00
	Bars, lb.	1232 @ 0.04 = 49.30	1025 @ 0.05 = 51.20
	Spiral, lb.	496 @ 0.05 = 24.80	168 @ 0.055* = 9.24
	Total	134.70	88.74

*$10 per ton is added to allow for the higher cost of cold drawn wire.

58 per cent for cross-sectional area, dead load and concrete
35 per cent for forms
17 per cent for bars
66 per cent for spirals
34 per cent for cost

In the estimate, $20 per ton is added to cover extra cost for high yield point steel and for accessories. This extra cost is probably too high. The unit cost for forms is taken as five cents per sq. ft. less for the 20-in. column in order to allow for simplifications due to standardization. This reduction will be discussed later.

For a 12-ft. story height, the total costs are $134.70 for the 31-in. column but only $88.74 for the 20-in. column. The difference is $45.96, or 34 per cent, a saving so great that it deserves the most careful consideration. Important but difficult to evaluate is the gain in rentable floor area for the 20-in. column which amounts to 3.89 sq. ft. per column. The advantages are of course less pronounced when column loads become smaller.

Column sizes are often limited by architectural restrictions, but the limit for principal columns is seldom less than 20 in. This gives an added advantage to the column with high yield point steel. Assuming other factors equal, the smaller size is always preferred.

STANDARDIZATION

Particular emphasis is laid on the fact that all columns carrying up to 1024 kips can be made just one size: 20 by 20 in. In this loading range, sizes of columns with intermediate grade reinforcement will vary all the way from 12 to 32 in. A multitude of sizes complicates

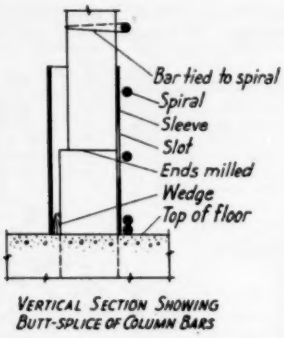


FIG. 2

both fabrication and erection of forms not only for columns but also for beams. It is better to maintain identical dimensions for the forms and to change the bars only.

Standardization of construction details ranks high in regard to economy. The word standardization is used here to mean making similar details alike. Many items can be enumerated in favor of making all columns one size. It is obviously more convenient both in architectural planning and in engineering design. In elastic frame analysis, for instance, the work is simplified when the column stiffnesses are known beforehand. The extent to which standardization can be carried is particularly significant in construction and erection. Forms are easier to build and may be used repeatedly without patching or rebuilding. Standardization, by using one column size throughout, will result in lower unit cost. As already demonstrated, the 20-in. column will also require smaller quantities.

DETAILS OF ERECTION

Returning to Fig. 1B, the 20-in. square column shown has 1½-in. protection and a 17-in. core. It has eight round bars in pairs and the minimum clear distance between bars in each pair is 4 in. This arrangement provides ample space for bars in beams both when the beams are centered on the column and when they are flush with one of its faces.

Bars may be spliced, say, three inches above the floor surface. It is recommended that only four alternate bars be spliced while the four intermediate bars be run through continuously. The bars should have milled ends and be kept securely in alignment.

The erection procedure is illustrated in Fig. 2. A 6-in. long pipe sleeve is wedged to the bar stub so that the sleeve bears against the

outside of the bar. The spiral is lowered and the four new bars placed in the sleeves. These bars are tied to the spiral with the result that upper and lower bars are aligned on the side toward the spiral. The sleeve shown has a slot through which bearing and alignment can be inspected.

After a floor is concreted, four bars at each column extend a story height above that floor. Forms are erected for the next floor, the spiral is slipped over the four bars already in place, and the four new bars lowered one by one into the column. A 24-ft. long 2-in. round bar weighs 256 lb. which is not considered too heavy to handle.

At top of the footing, an ordinary base plate may be set as in common practice for columns of structural steel. Instead of anchor bolts dowels may be used extending through a circular hole in the center of the plate. Bearing surfaces between bars and plate should be milled.

Splices in reinforced concrete columns are usually lapped but may occasionally be welded. Lapped and welded splices can withstand tensile stresses, but full tensile splices are not essential in reinforced concrete columns any more than in other types of columns. The A. C. I. Code specifies lapped or welded splices of "other positive connections." A more explicit recommendation is suggested patterned after a clause in the Chicago Code, 1937:

"Longitudinal bars may be spliced in any manner that will provide for all stresses that may be developed in the reinforcement."

This recommendation can readily be satisfied with splices as described in which one-half of the eight bars are fully effective in tension.

Bars with yield point stress of 75,000 p.s.i. may be procured either as a special alloy steel or by cold-stretching ordinary carbon steel. Either one may be used since the bars are neither welded nor bent. They should be deformed and should be identified by marking rolled into the surface during manufacture. The sizes that should be stocked are 1, 1¼, 1½, 1¾, and 2 in.

CONCLUSION

Other codes allow the same type of column to be designed with bars of high yield point stress. Among these are:

Joint Committee Code, 1940, Section 854 (a),
Pacific Coast Code, 1940, Section 2620 (c),
Chicago Code, 1937, Section 3902.09 (e).

The codes are identical in regard to the type of column discussed with exception of the Chicago Code. The maximum load allowed

by the Chicago Code on the 20-in. square column reinforced with eight 2-in. rd. bars is 1014 kips. The other codes allow a load of 1024 kips.

It is seen that the use of large bars with high yield point stress shows distinct advantages. Materials and costs are substantially reduced. For loads up to 1024 kips, a uniform column size of 20 in. may be adopted. This results in standardization and simplification in both office and field.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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Discussion of a Paper by Thor Germundsson:

Columns With High Yield Point Reinforcement Designed Under the A. C. I. Code*

BY F. E. RICHART, M. O. WITHEY AND BENGT FRIBERG

CONVENTION DISCUSSION

F. E. Richart†—Mr. Germundsson has presented three strong arguments in favor of this proposed type of column design. Probably the greatest is the matter of economy. In addition are the standardization and the possibility of reduction in column size and the elimination of excess steel at the column splice.

Disadvantages to be recognized include the extension of column bars through two story heights, even though it is proposed to stagger the bars so that half of them extend to alternate floor levels.

The proposed bar splice is reminiscent of practice of 30 years ago. Professor Talbot made tests at that time to study the effectiveness of butt splices in pipe sleeves, and it is interesting that this device is again proposed, with some improvement in detail. Mr. Germundsson has quoted requirements for splices prescribed in the new ACI Building Code. I believe the splices permitted under the Code were intended to provide a tensile as well as a compressive splice, though not specifically so stated, except for welded splices.

Welded splices, under the Code, should be butt-welded and should develop in tension at least the yield point of the reinforcing bar. I have been greatly impressed on construction where welding was available with the flexibility and usefulness of the process. I have also made tests of welded bars, and not only is it easy to develop the compressive strength of the bar in a butt weld, but the yield point of the

*ACI JOURNAL, Apr. 1941; *Proceedings* V. 37, p. 569; presented 37th Annual Convention.
†University of Illinois.

bar in tension can also be provided for. In fact, we made tests in which only one side of the bar was welded, with the thought that column bars might be accessible only from the outside of the reinforcing unit. By chamfering the bar ends and applying a V-fillet weld, it was possible to develop the tensile yield point of the steel in spite of the eccentricity thus produced.

It is evident that the butt splice saves a considerable length of bar, which would probably pay for a welded splice and much more than pay for the sleeve proposed in this paper. There may be some question with regard to the sleeve joint, since it provides for a tension connection in only half of the bars at each floor, whereas, whether recognized or not, there is often some bending stress in most building columns.

As I recall it, columns similar to those advocated by Mr. Germundsson were used successfully in a building at Rockford, Ill., some years ago. They utilized 2-in. square bars and, I believe, welded splices.

Mr. Germundsson suggests the use of a base plate at the bottom of a column with heavy bar reinforcement, with footing dowels brought through a hole in the center of the plate. In this connection the new ACI Code permits some variation from the common requirement for dowels. It permits some latitude in the number of dowels, requiring only that the total sectional area be at least equal to that of the column bars, that the number be at least four, and that the diameter of dowels be not more than $\frac{1}{8}$ in. greater than that of the column bars. Of course, in the usual case, the dowels should be as near the periphery of the column as other code requirements will permit.

*M. O. Withey**—First, I wish to compliment the author on the clearly and precisely written paper. In regard to the high yield point reinforcement, I have this comment to make: Necessarily there will be greater bond stresses required at the base of such columns and other places, and more care will be necessary in working out a safe design at those points than in the column of structural steel reinforcement. I am glad to see that the author has compared columns, but I think that there will be designers who will desire to make columns of high yield point steel with ties only, and for that purpose I think the paper would be strengthened if there were included a further comparison in which such a column was illustrated and its deficiencies indicated.

A. B. Cohen†—What would be the size of the sleeve?

*University of Wisconsin, Madison.
†Consulting Engineer, New York.

Mr. Germundsson—I think the sleeves would not have to fit very tightly. As long as they were not much too large, I think it would be satisfactory, so that you might be able to use one size of sleeve for two or three sizes of bars.

*William Flesher**—Do the bars have to be plain?

Mr. Germundsson—That is related to the point Professor Withey brought up, and I really cannot give an adequate answer. We have not dealt with such large bars and it is a thing that would have to be experimented with to some extent.

Prof. Withey—I do not think there should be any question about the adequacy of the bond. Hence tests should be made to assure that proper bond is developed.

Bengt Friberg†—I think there is no question about the desirability of the use of ingenious types of new columns for concrete. Entirely too long practice has been tied down to a type of column in which you put standard sizes of bars and provide a standard type of splice. For at least 20 years sleeve splices have been regularly used elsewhere even if not in common practice. Designing engineers generally should devise new and more economical columns in that manner. As far as the use of the 2-in. bars is concerned, it is a possible although not a generally available size of bar. I did not notice the price used in the cost tabulation for the 2-in. steel, so I do not know definitely the influence of the steel price upon the economy of that column.

Mr. Germundsson—Twenty dollars more a ton.

Mr. Friberg—Twenty dollars more; well, I would guess that price would be possible; with a substantial use, approximately that price would be possible also for 2-in deformed bars. The 2-in. bar however, equals two 1¼-in. square bars, and the bond area for the 2-in. round bar is 6.14 square inches per inch. The bond area for two 1¼-in. square bars laid together would be 7.50 square inches, not counting the contact area between the two bars. I believe that the column could be built with 1¼-in. square bars placed together and radially oriented in the column with even greater economy than the author mentioned. The construction involves consideration of what would happen to the unbonded contact area between the two bars. I think tack welding would not be required to prevent a cleavage plane—at least not in vibrated columns. Because of the greater bond area, bar deformations would be less essential. It would mean also that we could consider lap splice welding if the ends of the two bars were offset slightly.

*General State Authority, Harrisburg, Pa.
†Laclede Steel Co., St. Louis.

I have had no opportunity to see the paper before, or I would have prepared a sketch to show the construction. The main point remains: It is a good thing that designers should consider new forms of reinforced concrete columns and structures, and it can be done without radically new construction elements.

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The Design and Control of Paving Concrete in Iowa*

By Bert Myers†

SYNOPSIS

The paper reports Iowa Highway Commission practices over the last 20 years, in the design and control of paving concrete: (1) The design of proportions to make the most economical use of the aggregates available, particularly mixtures containing a high ratio of mortar to coarse aggregate; (2) the introduction of the practice of weighing aggregates for paving concrete; (3) improvements in methods for the control of thickness of pavement slab; (4) design of equipment for accurate control of measurement of mixing water; (5) detailed methods for control of proportions.

This paper reports the practices followed by the Iowa State Highway Commission for the last twenty years in the design and control of paving concrete. Since policies followed were greatly influenced by the kinds and quantities of materials available a few statements on this subject seem pertinent.

Practically the entire area of the state was covered by one or more of five continental glaciers. In most places the drift deposited by these great ice sheets covered the rock to depths of many feet. Subsequent erosion has not materially reduced the thickness of this cover in many places. Only the two most recent glaciers carried important quantities of sand or gravel and these two did not invade the southern part of the state. Thus deposits of glacial gravel are confined to the area covered by the two most recent glaciers and a few valley trains emerging from these areas.

The average composition of the glacial gravel deposits is about 35 per cent particles which retained on a No. 4 sieve and about 65 per cent sand. Most of this gravel contains considerable percentages of

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†Engineer Materials and Tests, Iowa State Highway Commission, Ames, Iowa.

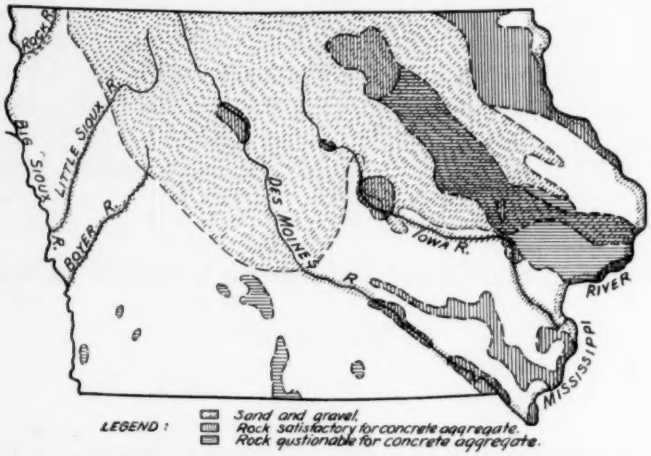

FIG. 1—SAND AND GRAVEL AND ROCK AREAS IN IOWA

soft and unsound particles, such as shale, ochres and iron stained concretions surrounding a soft or unsound center.

The glacial sands are very generally composed of strong durable particles. These sands seldom contain more than 5 per cent of particles passing a No. 100 sieve. Due to the sorting effect of running water the percentage of sand in the valley train deposits increases with the distance from the terminal moraine of the glacier of origin.

Except for a deposit of quartzite in the extreme north-west corner of the state, only a few acres in extent, the only rock available is limestone. Much of the limestone present at such depths as to make open quarry operation practical is so soft or so lacking in durability as to render it unfit for use as aggregate. Throughout most of the southern half of the state the rock present at reasonable depths is composed of such a low percentage of durable material interbedded with nondurable limestone and shale as to make the production of aggregate from it very expensive.

Fig. 1 shows the areas covered by the glaciers which deposited important quantities of sand and gravel and the important gravel trains which developed from them. It also shows the areas within which limestone is available with an indication as to whether it is suitable for concrete aggregate. These facts show that coarse aggregate is neither abundant nor well distributed; sand relatively more abundant and somewhat more widely distributed.

HOW TO DESIGN MIXTURES WITH MATERIALS AVAILABLE

This situation with regard to the availability of aggregates was recognized by those interested in highway construction before any

extensive road building program was started. Some time before 1919 the Engineering Experiment Station of Iowa State College began studies to discover how concrete mixtures should be designed to insure strength and durability and at the same time make the most economical use of the material resources of the state. It was obvious that to use the conventional proportions requiring about twice as much coarse aggregate as sand would involve large transportation expenditures and deplete the limited supply of coarse aggregate.

The use of the conventional proportions would affect the rate of depletion of coarse aggregate supplies for another reason that has not been mentioned. This is the fact that the pebbles in the gravel deposits are so predominantly of the smaller sizes as to require the wasting of a considerable percentage of the total quantity present to produce a grading adapted to use in mixtures containing the usual ratio of fine to coarse aggregate.

Therefore, under the direction of R. W. Crum there was carried on an extensive series of studies of the laws governing the strength of concrete and the application of these laws to the local problem. Some of the results of these studies were reported by R. W. Crum [1] in 1919. A more complete report is given in Bulletin 60 of the Engineering Experiment Station published in 1921.

In 1897 M. Feret [2] had presented the principle that for sand mortar there is a direct relationship between the strength of the mortar and its cement-voids ratio or $\frac{c}{1-d}$

Where c = the absolute volume of the cement present in a unit volume of the freshly mixed mortar and

d = the total absolute volume of the solid materials present in a unit volume of the fresh mortar.

The results of the tests made by Crum indicated that for the kind of concrete with which the study was concerned, the same principle applied to concrete.

These studies also resulted in the advancement of a theory as to the change which must be made in the cement-aggregate ratio to compensate for a given change in the per cent of sand in the total aggregate to maintain a constant cement-voids ratio and degree of mobility in the concrete. A statement of this theory is as follows: when the proportions are expressed in terms of pounds of aggregate to one pound of cement, for each increase of one in the percentage of sand in the combination of sand and coarse aggregate, the number of parts of total aggregate to one part of cement should be decreased by 0.05 parts. Conversely, for each decrease of one in the percentage

of sand in the total aggregate, the number of parts of total aggregate to one part of cement should be increased by 0.05 parts. An example:

Example. Assume that it is desired to determine the proportions in which aggregate composed of 50 per cent sand and 50 per cent coarse aggregate should be combined with cement to produce concrete having strength and mobility equivalent to that of concrete of the proportions 1:2:3.

Percentage of sand in the total aggregate of the proposed mixture = 50

Percentage of sand in the total aggregate of the given mixture $\frac{2}{2+3} \times 100 =$ 40

Proposed increase in percentage of sand in aggregate = 10

$10 \times 0.05 = 0.5$

Number of parts of aggregate to one part of cement in given mixture $2 + 3 = 5.0$

Minus 0.5

4.5

Therefore, the proposed mixture should contain one part of cement to 4.5 parts aggregate. Since the aggregate in the proposed mixture was to contain equal parts of sand and coarse aggregate the proportions are 1:2.25:2.25.

If the statement of the theory had been made in terms of absolute volume relationships rather than weight relationships it would have been more general in its accurate application. The theory does not take into account certain factors mentioned by Edwards [3] in his discussion of Surface Area nor certain factors mentioned by Abrams [4] in his discussion of the Fineness Modulus. Therefore, its application may be limited to sands having about the same grading, particularly in the finer sizes of particles. It does, however, provide a very quick and convenient method for adjusting proportions for the use of aggregates of any fine to coarse aggregate ratio under any conditions likely to be encountered under specifications requiring the sand to be reasonably well graded. The fact that its application requires the determination of the percentages of the aggregate passing a single sieve and the use of very simple computations makes it applicable to the proportioning of unscreened gravel or a combination of unscreened gravel and screened coarse aggregate.

At a time when pavements were being built in places remote from the few existing commercial aggregate processing plants and efficient portable screening equipment was not yet available, considerable mileages of pavement were built from concrete made with unscreened gravel or this material combined with screened gravel or crushed stone. In the Engineering Experiment Station Bulletin that has been referred to, mention is made of a pavement built in Palo Alto County in 1920 and 1921. This project, 6.9 miles long, was built of concrete made from unscreened gravel taken from a pit at the side of the highway. The material contained about 70 per cent sand

by weight. The proportions used were designed according to the theory outlined above and were intended to produce concrete with a strength equivalent to that of concrete of the proportions 1:2:3 ½ by volume. The success attained is indicated by the data shown in Table 1 in which the compressive strength of 6x12-in. cylinder specimens fabricated on 15 other jobs where the proportions were 1:2:3½ is compared with the strength of similar specimens fabricated from the concrete used on the Palo Alto County Job.

TABLE 1—COMPARISON OF COMPRESSIVE STRENGTHS OF 6/12 INCH CYLINDERS—AGE 90 DAYS

	No. of Specimens	Strength—Lb. per sq. in.		
		Max.	Min.	Ave.
Palo Alto County Project.....	98	6610	3210	4570
Fifteen Projects, Proportions 1:2:3½.....	715	7100	2015	4770

Due to the disparity in the numbers of specimens represented by the two groups of strength values a comparison of the differences in maxi-maximum and minimum strengths is probably not significant. However, the difference of only 200 p.s.i. between the averages indicates a remarkable degree of success in designing a mixture to produce the same strength as that of concrete of very different proportions.

This pavement built 20 years ago and other pavements built about the same time from concrete of similar proportions are still in good condition. They were built without joints and were reinforced only with two or three longitudinal bars and with transverse bars as much as 14 feet apart. It is true that pavements containing unusually large proportions of cement and sand developed transverse cracks at more frequent intervals, than pavements built of concrete containing a higher proportion of coarse aggregate. However, the presence of these cracks has not affected the utility of the pavements and it does not appear that their presence will affect their useful life.

CONTRACTOR CHOOSES AMONG VARYING PROPORTIONS

The Iowa Highway Commission has continued to provide in its specifications for the use of a number of proportions with varying ratios of sand to coarse aggregate, any one of which may be used at the contractors option. Provision is made for the use of unscreened aggregate also. These proportions are developed according to the original theory except that absolute volume relationships are used instead of weight relationships.

Since the importance of the grading of the coarse aggregate diminishes as the relative quantity of mortar in the mixture increases,

the grading requirements for the coarse aggregate to be used in the various mixtures are varied accordingly. For coarse aggregate to be used in a mixture containing a minimum quantity of mortar the limits for the percentage passing the various testing sieves are quite narrow. For a mixture containing a large proportion of mortar these limits are wide. Thus the mixture most economical on any given job will depend not only on the relative delivered cost of cement, sand and aggregate but also upon the relative delivered cost of acceptable coarse aggregate for each mixture. The producer of aggregates will sell at the lowest price the materials for the mixture involving the least waste. Table 2 shows the proportions specified for concrete to be used in pavement to be placed by vibratory methods.

TABLE 2—PROPORTIONS OF CONCRETE TO BE VIBRATED

% Sand in Aggregate	Basic Absolute Volume of Material per Unit Volume of Concrete			
	Cement Minimum	Water Approximate	Fine Aggregate Approximate	Coarse Aggregate Approximate
35	0.106620	0.155975	0.258047	0.479358
40	0.109446	0.160109	0.292178	0.438267
45	0.113794	0.166468	0.323882	0.395858
50	0.117952	0.172552	0.354748	0.354748

Table 3 shows a comparison of the average flexural strength values for the three most commonly used mixtures as determined from 6x6 in. beam specimens fabricated on all the paving projects built in the years 1932, 1933 and 1934. It will be noted that the greatest difference between comparable values is about 3 per cent.

MATERIALS BY WEIGHT

In the early 1920's the science of designing concrete mixtures had progressed beyond the art of producing these mixtures in the field. The importance of the accurate control of the quantity of each of

TABLE 3—AVERAGE FLEXURAL STRENGTH OF 6x6 IN. BEAMS
All paving built 1932, 1933, 1934

Sand, Per Cent of Total Aggregate by Absolute Volume	Flexural Strength—lb. per sq. in.			
	Age 7 Days		Age 14 Days	
	No. of Specimens	Strength	No. of Specimens	Strength
40	2769	677	2716	742
45	3019	666	2506	733
50	1102	657	1135	734

the materials entering into each batch was generally recognized and the effect of variations in these quantities was well understood. It seems strange, therefore, that a method as inaccurate as the volumetric measurement of aggregates continued to be used. It cannot be

said that engineers had failed entirely to recognize the advantage of weight measurement over volumetric measurement for in the 1905 edition of "A Treatise on Masonry Construction" by Ira O. Baker the following statement is made with reference to methods of proportioning mortar. "The most accurate but least common is to weigh the ingredients in each batch." The aggregates for bituminous mixtures had been measured by weight for several years. However, so far as this author knows the weight measurement of materials for paving concrete was not specified before 1923. Some time previous to that date George L. Campen found it necessary to mix some concrete in an asphalt paving mixer. Since the scales present in the asphalt plant made it inconvenient to install the ordinary volumetric measuring equipment the aggregates were weighed. For a few years the Iowa Highway Commission followed the awkward and inaccurate practice of computing proportions by weight and then transposing the batch weights into volumes for measurement in the field.

With some question in their minds as to whether weighing equipment adapted to use on a concrete paving job could be devised, what such equipment would cost, and whether the weighing of aggregates could be accomplished without delay to the work the engineers of the Iowa Highway Commission decided to require that the aggregates be weighed on one concrete paving project. The job selected was a project 17.7 miles long to be let on February 15, 1923 [5]. Experience on this job proved that weight measurement was entirely practical; that it was not expensive; and that it could be accomplished without delay to the work. In 1924 the requirement that aggregates be measured by weight was made a part of the specifications on all concrete paving work. This method of measurement is now widely used and its advantages are generally recognized.

REFINEMENT OF CONSTRUCTION EQUIPMENT AND PROCEDURE

With a method for measuring the dry materials in which he had some confidence the engineer became dissatisfied with the methods for the control of other factors affecting paving construction. So long as the quantity of aggregate entering into each batch was admittedly variable there seemed to be little hope of making a close estimate of the quantities of materials required to produce a given number of square yards of pavement of a given cross section. If these quantities varied from the assumed values the variation might be due to a variation in the bulking of the aggregates, to a variation in the amount of mixing water used or to a variation in the cross section of the pavement. The engineer and the contractor might argue about the

matter but they had little chance to make an intelligent correction. The accurate measurement of the solid materials focused attention on the effect of inaccuracies in finishing the subgrade.

It was found that with the tools available for finishing the subgrade the variation in the thickness of the pavement was likely to amount to several per cent of the designed depth. To improve this situation A. F. Miller and F. H. Mann designed the subgrade planner or drag template in 1924. The effect of the use of this tool riding on the top of the forms and capable of shaving the surface of the subgrade is indicated by the data shown in Table 4. These data indicate that the use of the drag template reduced the probable deficiency in slab thickness without requiring any great excess in thickness.

TABLE 4—ACCURACY OF CONTROL OF PAVEMENT SLAB THICKNESS AS INDICATED BY LENGTH OF CORES

Year Built	No. of Jobs	No. of Cores	Job Average Length of Cores Percentage of Design Depth of Slab		
			Min.	Max.	Ave.
1921	18	516	92.88	111.41	98.43
1922	9	372	95.88	100.63	98.13
1923	9	132	95.40	101.80	98.58
1932	46	1488	98.80	104.29	101.14
1933	32	832	98.57	105.37	102.06
1934	31	681	99.42	105.00	101.44

It was known that the moisture condition of aggregates as used might vary from that in which they would absorb a few per cent of moisture to that in which they contained several per cent of free moisture. While the weighing of these materials reduced the effect of these variations it was still sufficient to warrant a compensating adjustment in batch weights.

The specific gravities of aggregates used in Iowa vary from 2.53 to 2.70 or a variation of 6.7 per cent of the minimum. If the absolute volume of the aggregates in a batch was to be held constant the specific gravity of the materials used must be determined and corresponding adjustments must be applied. Therefore, soon after weight proportioning was adopted a device was developed for determining the specific gravity and the absorption or free moisture content of aggregates. This piece of apparatus consists of an ordinary glass fruit jar with a conical cap which has a small hole in the top. This pycnometer with an accurate balance provides a cheap, convenient, rapid and accurate method for making the necessary determinations.

CONTROL OF WATER IN BATCH

Accurate control of quantities of the solid materials in a batch of concrete for paving work was accomplished before a similar degree

of control of the water was attained. In 1928 the water measuring equipment of a paving mixer consisted of a single tank filled from a supply line at the side of the road. The equipment was designed to measure water by filling the tank for each batch and then withdrawing the water to a level predetermined to deliver the amount desired. As the tank filled air escaped from a valve located at the top of the tank and designed to close when the water reached it. The valves controlling the flow of water into the measuring tank and into the mixer drum were exposed to the full pressure of the supply line. Valves working under such pressures required constant and careful maintenance to avoid leakage.

It had been demonstrated that such water measuring devices properly designed to avoid inaccuracies due to tilting would function with great accuracy when connected to a source of supply such as a city water main. It was obvious that they failed to measure with equal accuracy under field conditions. In 1928 A. E. Stoddard [6] conducted a field study of water measurement on paving mixers to determine the degree of accuracy attained and the causes for inaccuracies. He found variations in supply line pressures as much as 150 p.s.i. occurring within short periods. Errors in measurement were as high as 7.28 gal. per batch or about 21 per cent of the intended volume. It was discovered that the chief causes of errors were the following:

1. The water supply contained varying quantities of entrained air which could not escape from the measuring tank.

2. Due to the high pressures to which the control valves were exposed they were frequently subject to considerable leakage.

The solution for both these difficulties was to discharge the water from the supply line into a storage tank not subject to pressure higher than that of the atmosphere and to supply the measuring tank by gravity from this storage tank. This offered opportunity for the entrained air to escape from the water and reduced the pressure on the valves controlling the flow into measuring tank and mixer drum.

Therefore, the requirement was made that water should be measured under atmospheric pressure. Fig. 2 shows a design for measuring equipment which complies with the requirement and which was recommended to contractors and mixer manufacturers. This design is similar in principle to a measuring device used by the Empire Construction Company on an Iowa Paving Job in 1925.

ARBITRARY PROPORTIONS

While the Iowa Highway Commission has pioneered in some phases of the design and control of paving concrete they have not followed

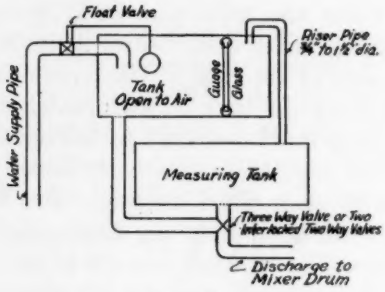

FIG. 2

the general policy of designing mixtures on a strength basis according to methods which take into account the concrete making properties of aggregates from individual sources. Under special conditions such designs have been made according to the Method of Talbot and Richart [7]. The method followed and the results obtained in two of such designs have been reported [8].

The reasons for continuing the policy of specifying arbitrary proportions are as follows:

1. There is some doubt that designs providing for concrete of the minimum strength commonly assumed to be adequate for paving work would in all cases provide concrete having satisfactory durability under the climatic conditions existing in Iowa.

2. It has been considered undesirable to disturb the aggregate market by changing the zones within which commercial producers can compete in delivered price.

3. It is considered important that the specifications provide information from which any prospective bidder can make an accurate estimate of the cost of the work. To provide such information under specifications requiring a specially designed mixture for material from each source of supply is difficult under conditions where much of the material may be secured from previously undeveloped supplies.

To provide the contractor with an accurate basis for estimating the cost of materials the specifications provide for a minimum quantity of cement per unit volume of concrete with the provision that the ratio of cement to aggregate will be changed if necessary to keep the cement factor within 1 per cent of that specified. This provision is made on the theory that the only source of variation in the cement factor from that assumed is a variation in the void content of the concrete from the assumed value. Thus if the void content is lower than that assumed the contractor is delivering a better quality of

concrete than that covered by the specifications. However, the decreased yield of concrete per batch causes an increase in most of the important items of cost. Therefore, the contractor is entitled to some adjustment. A slight increase in the amount of aggregate per batch will correct this deficiency in yield without any appreciable cost. Adjustments of this kind are made only after a careful survey to determine the actual yield of concrete per unit quantity of cement rather than the apparent yield obtained by assuming that the dimensions of the pavement slab are exactly the designed dimensions. Under these specifications the average quantity of cement used per square yard of pavement is about 102 per cent of the theoretical quantity. This excess of 2 per cent provides for all waste and for the excess depth of slab which cannot be entirely avoided under specifications requiring a specified minimum depth.

As indicated before, mixtures containing the maximum possible quantity of coarse aggregate and the minimum quantities of sand have not been designed. While such mixtures might result in temporary savings, they would also result in considerable waste of materials and the depletion of valuable natural resources and probably in an eventual increase in costs.

CONTROL OF PROPORTIONS

The details of the control of proportions have been fully reported in previous publications [9]. The following is a brief outline of this work. The control of proportions is in charge of an inspector working under the direct supervision of the Resident Engineer and the general supervision of the Engineer of Materials and Tests who has authority coordinate with that of the Construction Engineer. His duties are:

1. If the fine aggregate contains more than 5 per cent of particles retained on a No. 4 sieve, to compute the absolute volume proportions of the aggregate used to provide the specified absolute volumes of fine and coarse aggregate.
2. To determine the specific gravities of the aggregates.
3. To convert the absolute volume proportions into proportions by weight of saturated surface dry materials.
4. To determine the percentage of absorption or of free moisture in the aggregates.
5. To compute the batch weights necessary to provide the computed proportions on a saturated, surface dry basis.
6. To compute the total amount of free moisture in the aggregates in a batch and transmit this information to the inspector at the mixer to guide him in fixing the amount of mixing water per batch to be added at the mixer.
7. To check the accuracy and sensitivity of the weighing equipment.
8. To inspect the weighing of the materials.
9. To make strength tests of beam specimens molded from the concrete produced.
10. To report to the resident engineer, the district engineer and to the central office each day giving the results of tests made, batch weights used and the number of batches of each weight.

The science of the design and the art of control of mixtures to produce concrete of a desired density and strength have been developed to a high degree. It has been assumed that the durability of concrete was proportional to its density and strength.

If the destruction of concrete due to the effects of climate is due to the moisture absorbed this assumption seems logical. However, there is some evidence that the failure of concrete due to effects of climate may be similar to that of rocks. The geologist has observed that the rocks that show least resistance to weathering are the composite rocks. This suggests that the durability of concrete may be a function of the interrelationship of the elastic and thermal properties of its various constituents, the cement paste, the sand, the mortar and the coarse aggregate. Thus while the design and control of mixtures to produce concrete of a given desired strength has been interesting we may have even more interesting problems ahead.

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Discussion of a Paper by Bert Myers:

The Design and Control of Paving Concrete in Iowa*

BY R. W. CRUM, H. J. GILKEY, J. C. PEARSON, F. H. JACKSON,
E. F. KELLEY, LEWIS H. TUTHILL AND AUTHOR

CONVENTION DISCUSSION

Roy W. Crum† (at the conclusion of his presentation of Mr. Myers' paper)—It might be well to mention what I consider two notable features of this experience. One is the success of the engineers in making economical use of the most available materials under rather difficult conditions and securing a first class engineering result. Any one can take material from a distance, use it according to a conventional formula and get good results, but to take difficult conditions and find out how to do a good job is good engineering. The other point is the fact that a great deal of the success has been due to the type of men used as inspectors in control of proportioning, measuring and mixing concrete. They are practically all college graduates who have been put through an intensive course of special training. They have to be competent men who understand what they are doing or they are not able to handle such a complicated process.

H. J. Gilkey‡—Mr. Crum made one or two unpardonable omissions. Mr. Crum was Materials Engineer for the Highway Department, I believe, about the time this story started. I think these developments came into being about that time, and a great many of the men here know that probably no man in the country has had any more if as much to do with the weighing of materials for concrete as Mr. Crum. I always think of that as being perhaps Crum's greatest, among quite a number of contributions, to concrete.

*ACI JOURNAL, Apr. 1941; *Proceedings* V. 37, p. 577. Presented at 37th Annual Convention, in the author's absence, by R. W. Crum.
‡Iowa State College, Ames, Iowa.

Mr. Crum—I will admit that I had a little to do with the early stages of this work, but other engineers have been in charge for at least two-thirds of the engineering experience, and I think the credit goes to them and not to me.

*J. C. Pearson**—Mr. Myers' paper suggests a question about the Platte River gravel concrete, some of which I had an opportunity to examine in pavements in western Iowa last fall. Some of these pavements have gradually developed a peculiar type of cracking or disintegration, and I should like to ask Mr. Crum what the explanation, or the theory, of this type of cracking is.

Mr. Crum—I don't know, because I have not been acquainted with those pavements for nearly 15 years. The Platte River mixtures are about 30 per cent gravel and 70 per cent sand. The job in northern Iowa seems to be in perfect condition as far as the concrete is concerned. I understand that some defects have developed on pavements in which the Platte River material was used. Perhaps Mr. Jackson may have something to say on this situation. He has been in closer touch with it than I have.

F. H. Jackson†—I am afraid not. However, it is certainly a very interesting problem; one which has been developing for some years. The Public Roads Administration, among other agencies, is engaged right now in a rather extensive investigation to determine the answer. Whether the condition Mr. Pearson speaks of is due to some inherent characteristic of the aggregate, or whether it is due to the peculiar gradation of the material, is a question which must be determined. This material has a very unusual size distribution. There is normally an excess of medium size sand with a deficiency in minus No. 50 material. The maximum size does not exceed ½-in. It is really just coarse sand, and there have been serious troubles from the volume changes in that concrete. Just what the cause is, I do not know.

A Member—Gravel being an act of God, isn't there a possibility of a change in the characteristics of the gravel?

Mr. Crum—That Platte River material is composed of various materials, but the larger particles are predominantly of feldspar and granite with a large percentage of quartz while the finer particles are almost entirely of quartz. The material has a good record in sulphate and freezing and thawing tests.

E. F. Kelley‡—As an ex-Iowan, and just to keep the record straight, I might mention that the trouble is not confined to Iowa; the same

*Director of Research, Lehigh Portland Cement Co., Allentown, Pa.
†Public Roads Administration, Washington.
‡Public Roads Administration, Washington.

type of disintegration is taking place in Nebraska and Kansas with similar mixes. They are not always of Platte River gravel, and, as Mr. Crum has said, we know that certain of the mixtures that have been discussed have not developed that trouble. The trouble in Iowa, so far as our records go, is confined entirely to the western part of the state where either Platte River gravel or gravel with similar characteristics has been used.

*Lewis H. Tuthill**—In connection with this cracking, possibly the reason for the use of a higher water and cement content, is the higher sand percentage. It was stated a little while ago that the qualities of the Texas concrete were due to the low water content. I think perhaps the contrary has something to do with the conditions in Iowa.

Mr. Crum—It is not the cracking that is deleterious; the engineers out there do not worry about that at all, they say the cracking has not impaired the usefulness; the trouble is the disintegration of the concrete, as I understand it.

AUTHOR'S CLOSURE

In reply to Mr. Tuthill's remarks I would say that the water-cement ratio of the concrete made from the Platte River material is low. An average value is about 0.4 lb. of water per lb. of cement or 4.5 gal. per bag. Since the concrete contains 7.6 bags of cement per cubic yard the total water content is 34 gal. per cu. yd. of concrete. While this is somewhat higher than the water content of paving concrete used in some places it is not higher than that of great quantities of concrete that has shown no evidence of lack of durability. Concrete containing 6 bags of cement per cu. yd. and a water cement ratio of 6 gal per bag would contain 36 gal. of water per cu. yd. Mention has been made of the concrete pavement built in 1920 and 1921 in Palo Alto County using unscreened gravel containing about 70 per cent of particles passing a No. 4 sieve. While accurate records of the water-cement ratio of this concrete were not kept, records of the analysis of the fresh concrete indicate that it not less than 42.5 gal of water per cu. yd. of concrete. As Mr. Crum has mentioned, this concrete shows no indication of lack of durability after 20 years of exposure to rather severe climatic conditions. Therefore, it does not seem likely that the total water content of the concrete made from Platte River gravel and other materials which show similar behavior is a complete explanation of the trouble.

It is a serious matter when the aggregate most economically available in parts of four states produces concrete lacking in durability.

*Bureau of Reclamation, Denver.

It should be very uncomfortable for the Engineer to admit that he can find no way to produce durable concrete from such an aggregate particularly when it appears to be very durable in itself. However, that is the present situation with regard to some other aggregates as well as Platte River gravel. If this discussion which Mr. Pearson has introduced should attract the attention of some one capable of solving this interesting and important problem Mr. Pearson need not apologize for having asked his question.

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Technical Tedium or Otherwise*

By R. W. Crum†
MEMBER AMERICAN CONCRETE INSTITUTE

NOTE

Although this "brief dissertation on the art of presenting technical papers" (author's sub-title) loses in dramatic effect as a published record, it has fun mixed with solid substance. In response to an appropriate question Mr. Crum presented this paper at the "Quiz"—that novel (for A. C. I.) and final session of the Institute's 37th Annual Convention in Washington Feb. 18-20. With the author's hitherto little known histrionic talent the presentation was a "scream"—for all this contribution's serious intent. If you are going to present a paper (and who isn't?) read it.—Editor

At last your time has come; after hours of waiting through the interminable and boring speeches of your fellow conferees the chairman is about to call upon you. He announces your paper; you rise and, accompanied by a scattering of perfunctory applause, walk briskly to the platform, only stumbling on the top step and getting your feet tangled in the wires strewn around the floor for the loud speaker apparatus.

"Mr. Chairman, ladies and gentlemen," you say, and reach in your inside coat pocket for the paper. Consternation reigns for a moment when you find it isn't there, but you haven't quite lost all of your presence of mind yet and remember that you put it in your left hand outside coat pocket. Fishing it out you are ready to go ahead after first disburdening yourself of the few ill chosen introductory words that you‡ have been mulling over the last hour and a half.

*Received by the Institute, Oct. 21, 1940.
†Director Highway Research Board, Washington, D. C.; Member Board of Direction American Concrete Institute.
‡Any resemblance of the characters to any persons living or dead is entirely coincidental. The striking likeness of the first one to appear to the author of this paper is purely imaginary.

Your labored joke doesn't get much of a laugh but you are too flustered to bother about that. Grasping your bulky manuscript firmly but shakily in both hands and carefully avoiding the "mike", if there be one, you start to read. After reading one page you are somewhat appalled by the magnitude of the remaining task. You wonder momentarily if you are going to be able to get through in the allotted twenty minutes. But you haven't time to worry about that now and anyway time alone will tell. Gluing your eyes to the page you proceed. About this time some ill mannered person in the rear of the room calls "louder". Rather resentfully you raise your voice for a few lines but you can't think of that and keep your place at the same time so your voice soon fades down to your usual tone and thenceforward you pay no attention to such unwarranted distractions.

By this time you have recovered some of your self-confidence and most of the shakiness is gone and you are forging ahead full steam. But you don't dare glance at the audience or it will return and you will lose your place. On and on you go without thought of time. By and by you become aware that the chairman is trying to convey some sort of signal to you, but you can't take on his troubles too, and you keep on reading till you realize that you have come to that welcome period when you can show the pictures.

Saying, "I have some lantern slides that I would like to show if there is time". You start a search for the push button that will signal the operator, finding it, after some confusion, grasped firmly in your right hand.

The first slide comes on and turning your back to the audience you rather confidentially tell the screen what the picture is about. This takes some time as it is a complicated table with lots of data.* This part of the speech doesn't take over a half hour as there are only about a dozen of the slides crammed with tabulated data and intricate curves. In doing this you repeat practically everything you read before the picture came on.

As the lights return you notice that the chairman is about to have apoplexy and there is a red light shining on the reading desk in front of you, but the whole point will be lost unless you read your conclusions and so you struggle on manfully to the bitter end.

At last there is no more to read. You say "I thank you" and stagger from the platform with the beginning of a mild inner glow that says to you "well done". This feeling is heightened by the applause which

*The table was shown on the screen at this juncture—one of those tables so crowded with figures that its significant data (if any) dissolved into the all-over pattern of tight-woven homespun.

is louder this time, indicating that the audience's feeling of relief that you are through is slightly greater than its courtesy in giving you a hand when you started. To tell the truth, no one has listened for the last 45 minutes.

The chairman is now saying, "I am sorry that on account of the fact that we are way behind our schedule it will be necessary to defer the discussion of this interesting paper and proceed to the next item on the program."

You think, resentfully, that if the speakers before you hadn't taken so much time there might have been some interesting discussion of your absorbing topic. But you are too overwhelmed with relief at having the thing done to cherish that feeling long and settle down on the end of your spine to enjoy the next paper in a rather comatose condition.

But instead of being soothed by a gentle flow of monotone as you expect, something seems to be happening; your lethargy is being dispelled; the man on the platform is talking in such a way that you are becoming interested in spite of yourself. He is reading from a manuscript but from the way he is holding it you can tell it isn't very long. And he seems to know what he is saying so well that he can look up and actually talk to the audience most of the time.

It doesn't take him very long to tell what the paper is about and why and then he jumps right over the tedious details of what he did, which took so much time in your paper and goes on to tell what he found out. Soon he comes to a place where some illustration is indicated. Without pausing in his stride he signals the operator and a slide comes on the screen; just a simple little table that anyone can grasp at a glance but full of significance. Glancing over his shoulder to make sure the picture is the right one, the speaker describes it quickly and goes on to the next thought in his paper. Presently he comes to the next slide. This time it is a diagram; just a single curve, but illustrating a telling point forcefully and without loss of time.

Well, time has been getting on and the chairman even in this case signals the speaker that his time is about up. Strangely, this doesn't seem to disconcert him; he just goes for the last sheet of the manuscript, reads it quickly and makes a graceful exit. Too good to be true—for a burst of appreciative applause wakes you up and you realize that at least part of this remarkable performance has been the figment of a dream, but the man has done a good job and got through almost on time. Maybe he took the secretary's letter of instructions seriously.

Musing on this while sticking out the rest of the program you think, "I wish I could present a paper like that" and later, being a person reasonably quick on the intake and still having plenty of self-confidence you say to yourself, says you, "By heck, I can do a job like that." And so being a confirmed convention hound and a man of experience and ideas besides having opportunities to write technical papers, you make yourself over into a forceful and interesting speaker by means of a few simple principles based upon your illuminating flash of insight while drowsing through a dreary session.

1. You realize that most technical papers have to be long enough to cover the subject thoroughly with supporting data to satisfy the studious reader who will ultimately use your material, but you also realize that there is no need to present all this orally and that no one would get it even if you did. No one can get the good out of a technical paper by reading it just once. Many passages must be read and reread and reread and studied, so why should you expect anyone to grasp it all instantly while you read it to him. Better just read him the significant parts that he can assimilate as you go along.

2. It is apparent therefore that the first thing to do is to prepare a condensed version of the paper that can be read in the allotted time. Twenty minutes is about right, although some gifted persons can hold an audience with a dry subject for thirty.

In this version, tell what you did and why you did it and then jump right over the dreary details of how you did it and tell about what you found out that is new and interesting.

3. Having written the condensed version, read it aloud for timing. Then rewrite it so that the time will just be nicely filled. Then rehearse it—to your wife, or secretary, or the mirror until you can deliver it with some degree of animation and direct appeal to the audience. (It sometimes helps to pick out someone who looks as if he agrees with you and aim many of your remarks at him. His nods will spur you on, or else pick out a dumb looking one and see if you can wake him up).

4. But let's not forget the slides. If well done they can add greatly to the quick comprehension of a technical talk. Just remember to express only one idea on a slide and make it so simple that about all you need to do is name it. It is best to use the slides to illustrate points as you go along. If left to be shown all together at the end there is a great tendency to waste time repeating what has already been said.

5. Sometimes you do miscalculate and the chairman calls time on you before you are quite through. Why not provide in advance for

this contingency and if it arrives, just turn to the closing statement you have already prepared, read it and stop.

6. After you have prepared and delivered a few papers in this way you will probably discover for yourself that there is an even better way. Go ahead and write the short version, then write an outline of it, then memorize the outline and soak yourself in your subject till you don't need a manuscript. Just talk and follow the outline in the back of your head. Of course it is a good idea to have a copy of the outline on small cards in your pocket in case of emergency. You are not likely to need it but if you do, it will save the day. Doing it this way it is also easy to go too long and bring the chairman's warning down on you, so have that prepared closure all ready to grab and read if you haven't had time to memorize it.

7. This is not intended for a discourse on technical writing, but one principal should be mentioned that is of particular importance in preparing a short paper for oral presentation.

Write simply, using short common usage words and as few of them as possible.

For instance, to say: "When concrete is being placed during cold weather and the air temperatures may be expected to drop below 35 degrees Fahrenheit, a sufficient supply of straw, hay, grass or other suitable blanketing material shall be provided along the line of the work and any time when the air temperature is expected to reach the freezing point during the day or night, the material so provided shall be spread over the pavement to a sufficient depth to prevent freezing of the concrete before it has thoroughly hardened," may be perfectly good specification writing, but if you want to make the point in a speech why not say: "During freezing weather freshly laid concrete pavement should be protected by a suitable insulating layer."

On reading this thing over, it doesn't seem so difficult; I don't think one needs to be an expert to follow these rules. I think I will try it myself some day.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the* JOURNAL *Supplement concluding this Proceedings volume

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The Use of Carbon Dioxide to Reduce Efflorescence on Asbestos-Cement Shingles*

By Clyde R. Hutchcroft†
MEMBER AMERICAN CONCRETE INSTITUTE
and Harvey R. Anderson‡

SYNOPSIS

This paper describes experimental work with carbon dioxide gas to reduce efflorescence on asbestos-cement shingles. Factors investigated in connection with the reaction of carbon dioxide gas with the free lime of the shingles were: The effect of different arrangements of shingles for exposure to the carbon dioxide; the effect of the age and density of the shingles, concentrations of carbon dioxide gas, effect of temperature, moisture content and of the depth to which the reactions occurred.

Efflorescence or "bloom" appears on many portland cement products which are exposed to the weather. It has troubled manufacturers of asbestos-cement shingles for years. One would, of course, expect some trouble with efflorescence when it is characteristic of portland cement products; but that characteristic becomes much more troublesome with a product of various colors such as asbestos-cement shingles.

It is known of course that efflorescence results from the leaching of lime from the cement by the water absorbed during wet weather (especially so when the temperature is cool and favors the solution of the lime). Lime deposited at the surface of the shingle, is changed as it dries in the clear weather, to the carbonate by the carbon dioxide of the air.

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†Director of Research, Keasbey & Mattison Co., Ambler, Pa.

‡Research Chemist, Keasbey & Mattison Co., Ambler, Pa.

Many remedies for this trouble have been tried. The use of carbon dioxide to prevent efflorescence is one of the oldest and is probably still one of the most effective when properly used. We have attempted to improve the technique of its use to prevent "bloom."

Special apparatus was designed, including a source of carbon dioxide, a carbon dioxide heating unit, a chamber for treating the the shingles with carbon dioxide, and a fan to circulate the gas through the treating chamber. The treating chamber rested on a platform scale calibrated in one-quarter-ounce intervals. Water seals and flexible couplings were used to eliminate friction. Variations in temperature, pressure, and of the concentration of the carbon dioxide were determined by instruments placed in each end of the gassing chamber. Liquid carbon dioxide was used as the source of gas and was admitted to the apparatus through a pressure reducing valve. Fig. No. 1 shows the design of the gassing chamber.

The shingles for the experimental work were obtained from factory production, and with little time allowed between their manufacture and their storage in a closed container under humid conditions. While in storage for curing, the shingles were piled in such close relation to one another that only the edges of the shingles were exposed to any

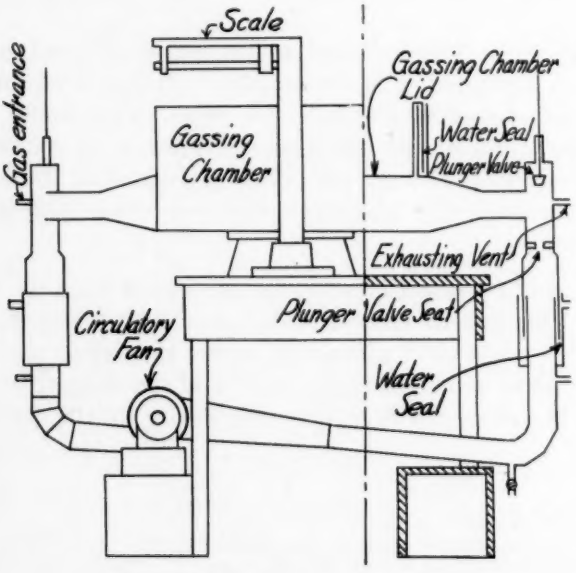

FIG. 1

air. Before the shingles were placed in the gassing chamber, strips two to four inches wide were trimmed from their edges. The shingles were separated by means of wooden spacers when placed in the gassing chamber, in such a way as to permit free exposure to the carbon dioxide while under treatment. The increase in weight of the shingles due to the reaction of the free lime with carbon dioxide was the measure of the reaction obtained. No further increase in weight was apparent when the reaction had reached completion. Likewise, no more gas was required to maintain the pressure in the apparatus.

To provide means of checking the effectiveness of the gassing treatment, an apparatus was used for producing an accelerated efflorescence. Shingles were immersed to half their length in water at approximately 60 to 70° F. for 16 hours for complete absorption. The immersed portion of the shingles was then rinsed in clear water and suspended in a humid atmosphere for eight hours at approximately 70°F. Shingles were then permitted to dry slowly with a gentle flow of air passing over them. The efflorescence appeared on the portion of the shingle which was immersed in the water. Ungassed shingles were run under the same conditions and were used for reference.

As a further check upon the effectiveness of the reaction, gassed shingles were checked for free lime at different depths in intervals of 0.005 in. by a modification of the glycerol-alcohol-barium chloride method for free lime as used by the Lehigh Portland Cement Co.

All experimental trials were made with a load of 24 shingles, approximately 12 by 14 in. so that the load would be of sufficient size to permit obtaining average results.

By means of the above described apparatus experimental work was done to determine: (1) The most suitable arrangement of the shingles for exposure to the carbon dioxide; (2) The rate of reaction of the carbon dioxide with the free lime in the shingles at various ages after manufacture; (3) The effect of concentration of the carbon dioxide upon the rate of reaction; (4) The effect of temperature upon the rate of reaction of the carbon dioxide with the free lime; (5) The effect of differing moisture contents in the shingle upon the rate of reaction; (6) The effect of shingle density upon the rate of reaction; (7) The depth to which carbon dioxide gas reacts with the free lime in the shingle.

It was learned at the beginning of the experimental work that it was not necessary to expose the back of the shingles to the carbon dioxide.

No benefit was derived by such exposure of the back side of the shingles. The treatment was, therefore, confined principally to the veneered surfaces. This was done by piling the shingles back to back with spacers to separate the face surfaces.

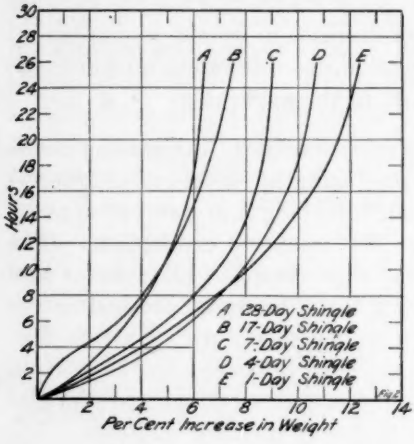

FIG. 2—EFFECT OF AGE ON REACTION WITH CARBON DIOXIDE

The age at which the shingles could be most effectively treated to prevent efflorescence was found to be from one to five days. Fig. 2 shows the extent to which the reaction could be made to proceed with respect to age. It will be noticed that the reaction was the greatest at early ages. The conditions of treatment, except for age, were similar for all the shingles here represented. From left to right the curves represent 28-day, 17-day, 7-day, 4-day and 1-day-old shingles. The initial deviation of the curve for the 17-day-old shingles was due to the concentration of the carbon dioxide being slightly lower for these particular shingles than for the others during the first four hours of treatment.

Fig. 3 was plotted from data obtained by gassing the shingles with all conditions constant except the concentration of the carbon dioxide. If the shingles were exposed initially to a high concentration of carbon dioxide, it was found that a greater amount of the free lime in the shingles could be carbonated than if the concentration was begun at a lower percentage and gradually increased. The rate of reaction was also greater when high initial concentrations of carbon dioxide were used. Curve A shows the result of treating five-day-old shingles with a carbon dioxide concentration of 12 per cent initially and gradu-

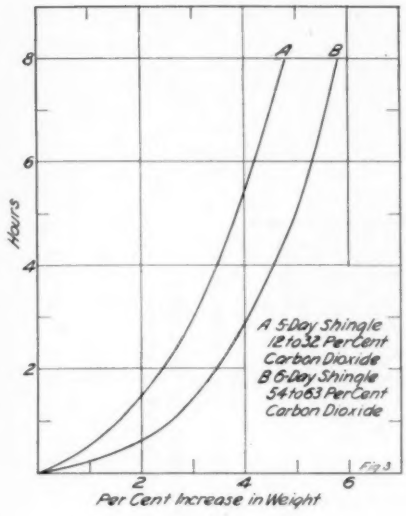

FIG. 3—EFFECT OF CARBON DIOXIDE CONCENTRATION ON RATE OF REACTION

ally building up to 32 per cent. Curve B shows the result on six-day-old shingles which were treated with an initial concentration of 54 per cent with the concentration gradually rising to 63 per cent.

Raising the temperature of the gas entering the gassing chamber above that produced by the reaction itself was of little or no benefit. That fact becomes apparent in Fig. 4. Shingles used for this test were gassed when 10 and 11 days old. Curve A indicates the result of gassing the 10-day-old shingles with the carbon dioxide entering the chamber at 154° F. Curve B shows the result when the carbon dioxide was allowed to enter the chamber at 88°.

Moisture content of the shingles did have a bearing on the reaction that could be obtained with the carbon dioxide. In Fig. 5, Curve A shows the result of thoroughly gassing of wet shingles; that is, the normal shingles containing 10.4 per cent moisture were thoroughly wetted with water before they were exposed to the carbon dioxide. Curve B shows the result of gassing shingles predried to a moisture content of 2.3 per cent. Curve C indicates the result of gassing the shingles with a moisture content of 8.3 per cent. It can be seen that the wet shingles showed a very slight tendency to react with the carbon dioxide. Shingles with a normal moisture content react at a much faster rate and to a greater extent than either the wet or comparatively dry shingles.

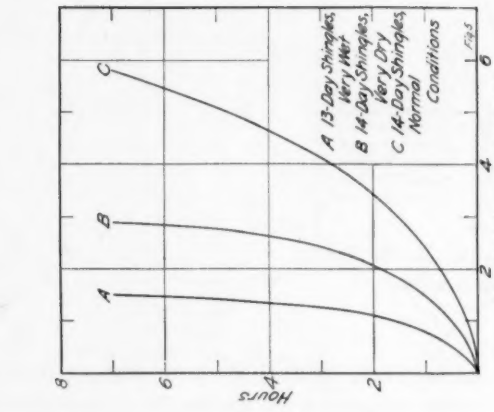

FIG. 5—EFFECT OF MOISTURE ON RATE OF REACTION

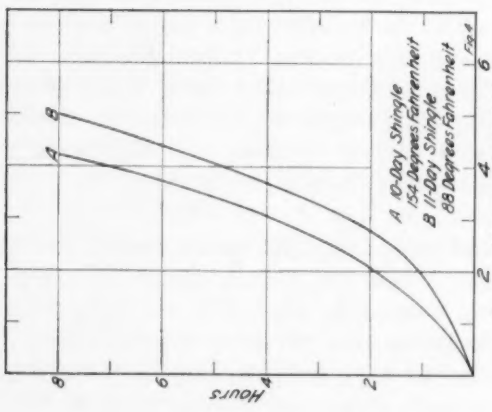

FIG. 4—EFFECT OF TEMPERATURE ON RATE OF REACTION

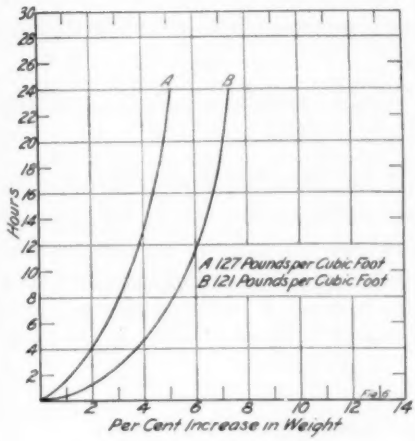

FIG. 6—EFFECT OF DENSITY ON RATE OF REACTION

It was apparent from the beginning of the work that the density of the shingles had a bearing upon the amount of reaction that could be obtained. The amount of carbon dioxide reacting with the free lime was less with shingles of high density than with shingles of lower density. For these runs all conditions except density were very similar. The results due to the effect of density are shown by Fig. 6. Curve A represents shingles having a density of 127 lb. per cu. ft. Shingles represented by Curve B had a density of 121 lb. per cu. ft.

In regard to the penetration of the carbon dioxide into the shingles to react with the lime, it was learned that a completely effective reaction did not go more than about 0.005 in. below the surface. The reaction had, of course, taken some effect below this depth, but it was continually less as it progressed toward the center of the shingle.

It was concluded that:

1. The shingles need be treated only on their face surfaces.

2. The shingles should be treated four to five days after manufacture.

3. The shingles should be exposed to a high concentration of carbon dioxide throughout the complete treatment.

4. The shingles should be treated at the temperature as produced by the reaction itself.

5. The shingles should be treated at a normal moisture content of eight to ten per cent with no unabsorbed mositure on their surfaces.

6. Shingles of a high density would react with less carbon dioxide than shingles of a lower density.

7. The reaction of the carbon dioxide with the free lime in the shingle was complete to a depth of about 0.005 in.

Treating the shingles according to the information gained from this work has not completely eliminated efflorescence, but it has greatly reduced this trouble. Conditions favorable to producing efflorescence are occasionally so severe that a few shingles on a newly applied roof may "bloom." Such instances have become a rarity, however; and of the many applications made in a year, only five or six roofs show any pronounced efflorescence.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1941*

Stucco Mechanically Polished on Columbia Broadcasting Studios*

By Walter B. Kaspareit†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

In modernizing the street facade of the Columbia Broadcasting Studios, New York City, face brick was cut back and three-coat stucco applied, the finishing coat being carborundum-ground to a smooth finish. This facing avoided removing the entire front of the building; saved time and cost. The materials and methods are described with suggestions for their improvement on other similar jobs.

Among several new types of stucco, all of them formulated with portland cement, is a mechanically carborundum-ground, smooth, glass-like stucco finish which was used on the exterior of the new Columbia Broadcasting Studios in New York City.

The Studios are housed in a remodeled 8-story building 75 ft. wide and 100 ft. high; originally faced with face brick. Many types of materials with which to reface the building were considered by the architect and the CBS engineers. After thorough investigation, specifications were written calling for three coat portland cement stucco. The decision to use stucco was based on the following:

1. It eliminated the need for removing the entire front of the building.

2. It permitted the use of a plastic material which would not extend beyond the building line. Use of other surfacing materials such as terra cotta, glass, or architectural concrete slabs would have made it necessary to cut the brick back to a greater depth than that required for stucco. This would have weakened the structure, added

*Received by the Institute Jan. 27, 1941.
†Vice-president Artstone Rocor Corp., Brooklyn, N. Y.

to the cost, and presented the problem of adequate anchorage to the cut-back brickwork.

3. The time element was important. The use of stucco assured completion of the job within the time specified. It would have been necessary to detail and burn terra cotta, the planning and manufacture of architectural concrete slabs would have consumed too much time, and in the case of glass, it was found that the building code of New York City prohibited its use on a building of this height.

4. The cost of stucco was from a quarter to a third that of any other material which might have been selected.

When the specifications were written a floated finish coat of stucco was considered the most desirable. At best, however, this finish was subject to soiling from floating dust and dirt particles, gas fumes, smoke, etc. It would be difficult to clean such a finish. It may be desirable to point out that stucco cannot be left smooth-troweled without subsequent treatment, since such surfaces usually show ugly trowel stains. These stains normally are removed with acid. Acid treatment, however, leaves the surface slightly pitted to collect dust. The architect and the client desired a semi-glazed finish that would tend to repel moisture and dirt and that could be cleaned easily with soap and water.

Tests were made and it was found that by grinding the stucco surface mechanically, using carborundum discs, a smooth, glass-like surface could be achieved economically. Test panels were made and the method accepted by the architect and the CBS engineers.

PREPARATION OF SURFACE

The original structure was trimmed of all ornamentation, projections, window heads, window trim, etc., and the entire surface roughened to assure a bond for the scratch coat of stucco. The many windows were bricked in with common brick, and mortar joints were raked back. The front of the building was divided by thin zinc parting strips into horizontal bands, approximately ten feet apart. These extended across the width of the building, and were securely fastened to the brickwork. This permitted the plasterers to work to natural breaks. On the completed job, the strips are hardly noticeable.

MIX FOR SCRATCH AND BROWN COATS

The mix for both scratch and brown coats was one part portland cement and three parts sand with 15 per cent lime putty (by volume). The scratch coat was applied over old brick, new brick, and limestone. It varied in thickness due to the unevenness of the roughened surface, and the average thickness was over ½ in., even more on lower panels.

The mortar for both coats contained metallic stearate to reduce suction for each succeeding coat. While it was intended that both scratch and brown coats be cured with water and be permitted to dry out for not less than five days, there was insufficient time to do this on the lower panels. Brown coat was applied to an average thickness of ⅜ in. Each base coat was cross-scratched.

MIX AND PLACEMENT OF FINISH COAT

The Artstone finish coat* consisted of Atlas White portland cement, dolomitic marble and silica aggregate, metallic stearate, inorganic colloidal plasticizer, and metallic pigment. No lime was used, as tests indicated that too long a period would have to elapse before the surface could be ground. The finish coat was applied over the dampened brown coat from break to break, *i.e.*, between the zinc parting strips. It was floated and then lightly troweled to imbed the aggregate.

On the CBS building the finish coat was approximately ¼ in. thick. Grinding removed approximately 1/16 in. of the surface and the few trowel stains. Time did not permit a five-day wait between the brown and the finish coat on the lower panels, but as much time was allowed as was possible consistent with completing the job within the time specified.

GRINDING MACHINE

Tests had been made with machines with swinging arms such as are used for grinding terrazzo wainscots, but these were unsatisfactory. The machines finally selected† were comparatively light, driven by a ¾ h. p. motor, and equipped with a seven foot flexible shaft to which, on a spindle, was attached the circular carborundum block. Water valves on the spindles assured a continuous supply of water. Both No. 40 and No. 80 carborundum blocks were used. A speed of 860 r. p. m. worked out best.

The surface was kept wet during the grinding process, and the workmen used a parallel, horizontal movement, until a smooth, glass-like surface was obtained. Smooth to the eye and to the touch, the exterior of the building today appears to be as clean as when finished eight months ago.

The job has aroused a great deal of interest among architects, engineers, and building owners, and has occasioned much favorable comment. Though not perfect, it should help to open the market for a smooth, glass-like, stucco finish that will tend to repel dirt, a particularly advantageous feature in metropolitan areas.

*Artstone Rocor Corp.
†Mall Tool Co., Chicago.

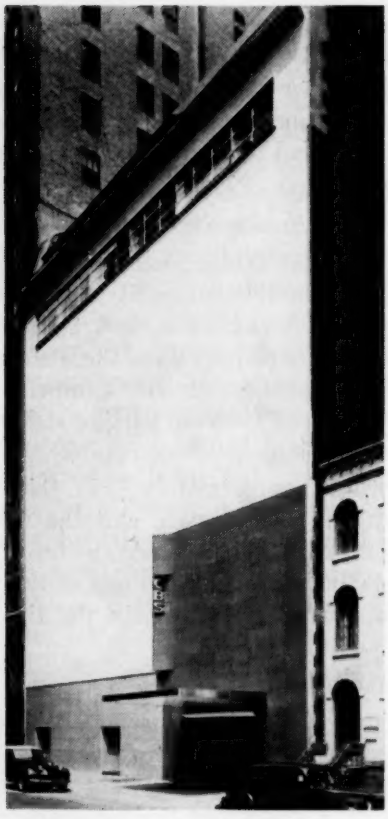

FIG. 1—CARBORUNDUM-GROUND STUCCO ON COLUMBIA BROADCASTING STUDIOS, NEW YORK

FIG. 2—THE GRINDING OPERATION

The extra cost over standard three coat portland cement stucco was less than 35 cents per square foot. This includes extra labor for troweling and grinding the surface, the cost of horizontal zinc parting strips, and the prepared stucco for the finish coat. It is well to point out, however, that it should be possible on future work of this type to cut costs appreciably.

RECOMMENDATIONS

On future work of this character the following recommendations are made:

1. Scaffolding should be placed in relation to expansion joints or parting strips. This would avoid all possibility of joining marks in application of finish coat stucco and in later grinding of the surface. The position of the scaffolding on the CBS job made this difficult. Where correctly formulated prepared stucco is used, joining marks

tend to disappear in time, but they are annoying at the outset and can be avoided.

2. Because the work had to be hurried, the scratch coat of stucco on the lower panels was applied over the still green mortar of the new brickwork and in a coat, in some places, approaching one inch, and even more. On remodeling work of this character, ample waiting time should be allowed (a) for fresh brick mortar to set firm and hard so that shrinkage takes place before scratch coat of stucco is applied, and (b) for the application of a sub-scratch coat where the scratch coat is likely to exceed ½ in. in thickness. Application of this extra coat, of course, costs both time and money, but since a considerable saving in both time and money is assured by the use of stucco in any event, proper allowance should be made for both.

3. There would be a marked advantage if treatment of finish coat of stucco could be conducted in the manner used for a cement floor finish. Suction, of course, would have to be under control. This could be accomplished by keeping the brown coat continuously wet for an hour or so before the finish coat is applied, and immediately before its application sponging off excess water. The stucco finish coat then could be properly floated to a true and even surface. First troweling would not have to be hurried and a second, and even a third, troweling could follow. A badly trowel-stained surface would likely result, but these stains would be removed in the grinding process. This method would achieve a more compact surface better for grinding than that obtained where the troweling merely imbedded the aggregate in the still plastic floated surface.

4. Lack of experience in grinding an unbroken area of this size, 4428 sq. ft., led to some difficulties. Work should not be hurried and under no circumstances should the work of grinding proceed until the stucco is hard. Five days would appear to be the proper time to wait before grinding the finish coat. Exact determination of time to begin grinding will depend on suction of base coats and on weather conditions. It must be remembered that if too much time elapses, the work of grinding the surface becomes more difficult and more costly.

5. A better method for grinding the surface should be developed. Guards or guides, perhaps, should be provided for the rapidly revolving carborundum block so that the workman will hold it squarely against the wall at all times. If the block is tilted, a gouge is made which is not easily repaired. Then, too, such a device might help to prevent the slightly undulating surface as on the CBS job. However,

this is noticeable only in cross sunlight, and is not regarded as a particularly objectionable feature.

CONCLUSION

Work of this character promises to create new interest in stucco. A stucco finish that possesses many of the advantages of other more costly surfacing materials should warrant increasing interest. Architects and engineers are always seeking materials that can be applied at low cost. For many purposes, and especially where large, unbroken areas are wanted, mechanically ground stucco may prove to be the answer.

The CBS job is not perfect. Further work and study are needed, but since much was learned from this first job, and since there exists a tremendous modernization market in all urban centers, there is a definite incentive for those who will most benefit to devote both the time and the effort to opening up this field for this new type of stucco finish.

In conclusion, I wish to express my grateful appreciation to Alfred Fellheimer of Messrs. Fellheimer & Wagner, architects for the CBS job, F. L. Hewes, the plastering contractor, John Buonaccorsi, under whose direction the work of grinding the stucco was done, and to the Universal Atlas Cement Co. for their kind and helpful assistance in the preparation of this paper.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by July 15, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1941*

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Color characteristics of cement

L. R. DAWSON, R. V. ANDES and T. D. TIEMANN. *Industrial and Engineering Chemistry*, Industrial Edition, Jan. 1941, Vol. 33, No. 1, pp. 95-98. Reviewed by ROY N. YOUNG

A Zeiss Pulfrich photometer was used for quantitatively determining color characteristics of many portland cements including three white cements and a number of other substances. Among the normal cements the reflectivity (lightness) varied from 15.0 to 35.1, and for the white cements it was of the order of 75 as compared to 100 for a baryta plate. Greater variations were found in the percentages of red and blue than green and yellow but the range of color characteristics was relatively small. No definite correlation could be drawn between the composition of the cements and color due to masking effects of burning and cooling treatment. The general effect of decreased Fe_2O_3 content or increased specific surface is an increase in reflectivity.

Precast supports and girders carry 4-acre reservoir roof

Engineering News-Record, Vol. 126, No. 1, Jan. 2, 1941, pp. 36-37. Reviewed by S. J. CHAMBERLIN

Twenty-four-foot columns were erected by a crane which set the dowel into the socket of the piers. Reinforcing steel at the top of the column was left projecting to be welded later to steel bars extending from the ends of other precast members that meet on top of the columns. After welding the joints were encased in concrete poured in forms set up around and bolted to each column. Bridging and enough transverse beams for rigidity were poured in place first. The 8-in. transverse beams being cast integral with the concrete poured over column tops to connect adjoining girder ends. All purlins were cast with a nailing strip for the galvanized metal roofing.

Variables of sugar test

GEORGE WILEY, *Rock Products*, Jan., 1941, Vol. 44, No. 1, pp. 126-127. Reviewed by ROY N. YOUNG

The Merriman sugar test for cement was investigated in connection with the effects of several variables introduced with the cement. It was found that the solubility index of the clinker tested was greatly increased by increased specific surface within the range of approximately 1200 to 2500. The greatest effect occurred at

specific surfaces above 2000. The solubility index was lowered markedly by the addition of water to the clinker just before grinding and also by the absorption of water by the cement after ground. In both cases the index was lowered from about 50 to 3 by the addition of 0.5 per cent of water. It follows that the sugar test is not practical as a test for underburning.

Precast box beams for high strength

HOMER M. HADLEY, *Engineering News-Record*, Vol. 125, No. 25, Dec. 19, 1940, pp. 70-71. Reviewed by S. J. CHAMBERLIN

Report of tests on four concrete box beams. 36 by 10½-in. deep in cross-section, the beams consisted of a 3-in. top slab, 1½-in. bottom slab, 2 and 3-in. longitudinal webs and 2½-in. cross webs. Large fillets made the unbraced length of thin sections not greater than 6-in. either horizontally or vertically. "Bundle-bars," each consisting of four ½-in-sq. bars constituted the principal reinforcing. On a 19-ft. 4½-in. span, failure was produced by a concentrated load at the center of 60,000 lb. A 6-ft. span carried 160,000 lb. The beams would be placed side by side to form a bridge deck.

Field-spliced precast concrete arches

JOSEPH J. WEILER, *Engineering News-Record*, Vol. 125, No. 21, Nov. 21, 1940, pp. 56-57. Reviewed by S. J. CHAMBERLIN

In building a fireproof garage for road machinery precast arch frames and roof slabs of light-weight concrete were used. To facilitate shipment and erection each 40-ft. span frame was made in three sections, with joints near the inflection points of the horizontal member, and spliced after erection. With the three units lined up, the four outside corner rods of the arch reinforcement were butt-welded and a ⅜-in. U-stirrup welded to the main steel on the center line of the splice. The joint was then filled and tamped with concrete. Hollow 6-in. roof slabs were bedded in mortar on the frames. Tests of the splice with a simple vertical joint showed strengths 3½ times the design shear.

Forming and concreting sewage tanks

Engineering News-Record, Vol. 125, No. 15, Oct. 10, 1940, pp. 51-53. Reviewed by S. J. CHAMBERLIN

Ingenuity and advanced planning produced an economical setup on forms and concrete placing for large diameter circular tanks and for aeration tanks at New York's Jamaica sewage plant. The aeration tank forms were practically all No. 1 spruce except the lagging, which was fir, covered with 3/16-in. oil-tempered fiberboard. The 4-ft. panels with 5 studs each and held by two ties at each wale location were stripped and moved over the completed base with a hand hoist. Double 2 x 4 wales were added continuous along the 42-ft. length of block. For the conduits the invert form was left in place and on it was set a pre-fabricated semicircular section made up of 1¼-in. spruce lagging covered with 24 gage black sheets. The top section dropped out, permitting easy stripping. The forms for circular tank walls were of ⅝-in. plywood panels prefabricated and marked for exact location with 3 by 4-in. studs. Four-inch channels were bent accurately to the tank radius and used as inside and outside wales. The concrete was distributed by pipeline to most of the work. For the circular tanks the pipeline was supported by a gin pole and the concrete carried to a center swiveled chute whose outer end was mounted on a rubber-tired carriage.

Good concrete stays good in New York seawater

L. C. HAMMOND, *Engineering News-Record*, Vol. 125, No. 25, Dec. 19, 1940, pp. 64-66.
Reviewed by S. J. CHAMBERLIN

A survey of harbor structures at New York City showed that, in spite of the severe disintegration noted in some of the structures, concrete can and does resist the ravages of seawater over long periods when it is properly designed, mixed, placed, cured and subsequently protected. Dense, rich mixtures are required and concrete in the tidal range needs to be protected by wood or masonry facings. Results of the survey were used as a guide in building a new project. The materials were carefully selected, the mix designed for high strength and density; and 6¼ bags of sulphate-resistant cement per yard specified. The sand was well graded and required to contain from 3 to 7 per cent of fines. All steel was covered with at least 2½ in. of concrete, and strict inspection was maintained.

Determination of surface areas

P. H. EMMETT and THOMAS DEWITT. *Industrial and Engineering Chemistry*, Analytical Edition, Vol. 13, No. 1, pp. 28-33. Reviewed by ROY N. YOUNG

In recent years a relatively simple and rapid method has been developed and used for determining the surface areas of metallic catalysts and non-metallic absorbents. This paper gives an account of an effort to utilize this method in measuring surface areas of industrially important finely divided materials. It is based on the absorption of a monomolecular layer of some gas such as nitrogen by the material. The number of molecules absorbed multiplied by the average area occupied by each molecule gives the absolute area of a given weight of the powder. A cement which had a surface area of 1890 sq. cm. per gram by the Wagner turbidimeter gave a value of 10800 sq. cm. per gram by this method. The authors preferred not to attempt to explain this large discrepancy but suggested that the error probably should be looked for in some of the assumptions made in elutriation methods.

Exposed interior concrete used for high class apartment house

Engineering News-Record, Vol. 126, No. 9, Feb. 27, 1941, pp. 43-44.
Reviewed by S. J. CHAMBERLIN

The columns, beams and underside of the floor slab were left exposed. All floors were identical and the columns were of uniform size for the full height, 3000 p.s.i. concrete being used for the three lower stories and 2500 p.s.i. concrete for the upper frame. There was a complete set of forms for one full floor. One transverse construction joint assisted in scheduling the construction work to keep all trades continuously busy. Deck and column forms were of ⅝-in. douglas-fir plywood. A medium slump concrete with ¾-in. maximum size crushed stone aggregate was used. Flexible-shaft vibrators assisted in compaction. Only minor rubbing was required before applying the casein paint (2 coats). Interior walls were of cinder block, plastered and painted. Exterior walls were non-loading bearing and consisted of two courses of brick separated by a 2¼-in. air space.

Plant for Denison Dam outlet works

E. G. WERNENTIN, *Engineering News-Record*, Vol. 126, No. 9, Feb. 27, 1941, pp. 47-49.
Reviewed by S. J. CHAMBERLIN

Two sizes of crushed stone, unloaded from hopper bottom cars, and the sand, from end dump trucks, fall into a track hopper and are fed by a 30-in. feeder belt

to a 36-in. belt conveyor, thence to a 24-in. shuttle conveyor which distributes the material into stock piles. From the stock piles the materials are fed by double clam gates to a second 36-in. belt conveyor located in a tunnel beneath the stock piles which carries the aggregate to the bins at the concrete plant. A screw conveyor and bucket elevator raise the cement from the storage silo to the plant cement bin. The fully automatic concrete plant of 500 tons capacity is located just outside of the finished structure. The concrete from the two 2-cu. yd. tilting mixers is carried in a 4-yd. shuttle car operating over a trestle and discharged by gravity from the car to the cableway buckets. A 1660-ft. traveling cableway covers an area 256 ft. wide by 1632 ft. long.

Effect of heat treatment and cooling rate on the microscopic structure of portland cement clinker

GEORGE W. WARD, *Journal of Research* of the National Bureau of Standards, Vol. 26, No. 1, p. 49, Jan., 1941. AUTHOR'S ABSTRACT

Ten commercial portland cement clinkers with their slowly and quickly cooled counterparts were examined microscopically. Methods are given for the conduction of the microscopical examination.

The appearance of the clinker phases after the different heat treatments is described and the structure of the clinkers discussed.

In general, for all clinkers, the phase compositions as calculated with consideration of glass agreed better with the microscopically determined phases than did those based upon the calculations in which complete equilibrium crystallization was assumed. The microscopically determined $3CaO \cdot SiO_2$ was nearly always greater and the $2CaO \cdot SiO_2$ lower than that calculated by either of the other methods. The $3CaO \cdot Al_2O_3$ was always less than that calculated from chemical analyses. Generally the $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ found microscopically exceeded that calculated by either of the other methods. The glass determined microscopically and the values obtained from the heat of solution method agreed better for slowly and plant cooled than for quickly cooled clinkers.

Hydrothermal and X-ray studies of the garnet-hydrogarnet series and the relationship of the series to hydration products of portland cement.

E. P. FLINT, HOWARD F. MCMURDIE, and LANSING S. WELLS, *Journal of Research* of the National Bureau of Standards, Vol. 26, No. 1, p. 13, Jan. 1941. AUTHOR'S ABSTRACT

Although rather extensive investigations have been made on the reactions of the crystalline constituents of portland cement with water, no such study has been reported with respect to the glass phase in cement clinker. In the course of some experiments related to this problem, on the reaction of water at elevated temperatures and pressures with glasses in the quaternary system CaO-Al_2O_3-Fe_2O_3-SiO_2, certain results were obtained which led to an investigation of possible solid solution relationships between the isometric tricalcium aluminate hexahydrate and the isometric tricalcium ferrite hexahydrate recently discovered by Eiger. The existence of such solid solutions was confirmed, and it was found that their stability was considerably influenced by the presence of silica.

An extension of the study revealed that silica could replace water in both $3CaO \cdot Al_2O_3 \cdot 6H_2O$ and $3CaO \cdot Fe_2O_3 \cdot 6H_2O$, and that the end products of these substitutions are grossularite garnet, $3CaO \cdot Al_2O_3 \cdot 3SiO_2$, and andradite garnet, $3CaO \cdot$

$Fe_2O_3 . 3SiO_2$, respectively. Complete solid solutions exist between each of these compounds and the other three. The hydrous members of the series may be termed "hydrogarnets" to indicate their relationship to the naturally occurring garnets.

Paving scaling successfully checked

O. L. Moore, *Engineering News-Record*, Vol. 125, No. 15, Oct. 10, 1940, pp. 61-64.
Reviewed by S. J. Chamberlin

Laboratory tests on small slabs made of various portland cements, blended cement, and special laboratory cements, showed that entrained air had something to do with prevention of scaling. Slabs containing small amounts of wood resin, fish-oil stearate or cod liver oil did not scale after many repetitions of ice-salt treatment. A test road of successive 20-ft. lengths, made of different types of concrete, was constructed as a field check of the laboratory results. One corner of each length was diked in winter and subjected to repeated freezing followed by a heavy application of calcium chloride to thaw the ice. Different cements, two sands, and medium and wet consistency were used.

Resistance to salt-scaling was attained by using cement containing very small amounts of fatty or resinous materials. Fineness of the cement had little effect, and amount of mixing water, although important, was not the major factor. The fatty or resinous material entrains air during mixing, practically eliminating segregation and bleeding. The reduction in strength is somewhat counteracted by the improved workability which allows a decrease in water content. The deleterious effect of inferior sand is reduced when cement containing fatty or resinous material is used. The scale resistance of certain natural cements is due to the fat or grease found in these natural cements. Portland cement when ground with the optimum of scale-preventing additions gave results superior to those obtained with the natural-portland blend.

Effect of aeration of cement on its properties

Kosaburo Watanabe (Ube Cement Co., Ltd.) Published by Association of Japanese Portland Cement Engineers, Osaka, Japan, p. 33. Highway Research Abstracts

The properties of portland cement undergo some change when it suffers aeration owing to access of moisture and carbon oxide during storage. In this study three kinds of cement—normal, rapid hardening and high silicious mixed cement—were investigated. The report includes both the chemical analyses of the cements and the calculated content of the chief compounds.

Samples of each kind of cement packed in paper sacks and sealed tin containers were piled in a column on wooden bars in the temporary store house from January 18, 1939. After 1, 2 weeks, 1, 3, 6, 9 months storage, the stored cements in sack and container at the top of the column were tested. The effect of aeration was determined by comparing the test results of the cement stored in sacks with that in sealed containers. By this process the influences of external conditions—temperature, moisture, etc.,—on the results of tests were eliminated.

The general conclusions from this work are: (1) Setting time of normal or rapid hardening cement increased up to 2 weeks aeration and later decreased but even at 9 months aeration it was longer than the fresh cement. The setting time of high silicious cement increased with storage. (2) Workability—showed by slump or flow—decreased when normal or rapid hardening cement was stored for more than one month but increased for high silicious mixed cement. The value of slump test

was more influenced by aeration than that of flow test. (3) The effect of aeration was greater on compressive strength than on tensile or bending strength. (4) Aeration first affected the strength at early ages then at later ages. Generally the reduction in strength was greater at early ages than at later ages. (5) The strength of 1:3 mortar was least affected by aeration and its effect on the strength of concrete and wet mortar was almost the same. (6) After 6 months storage cement lost about 30-40 per cent of its compressive strength when tested at 3 days and 20-30 per cent when tested at 3 months as compared to the strength of the fresh cement. (7) Among these three kinds of cements the rapid hardening cement was the most resistant to aeration and the high silicious mixed cement the least but the difference was not great. (8) The strength of high silicious cement began to decrease earlier than the two others. (9) The ignition loss of the normal cement increased from 0.84 to 3.20 per cent during 9 months storage; of the rapid hardening cement from 0.64 to 4.02 per cent; and of the high silicious from 2.08 to 4.14 per cent.

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The Use of Absorptive Wall Boards for Concrete Forms*

By W. R. Johnson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

A summary of laboratory tests and field observations at Kentucky Dam on the use of highly absorptive wall boards for concrete form linings. The use of absorptive wall boards is a large scale application of the principles practiced by John J. Earley in his use of absorptive plaster molds for architectural concrete. Absorbent forms eliminate voids and air pockets on concrete surfaces but this is of secondary value when compared to the greatly increased quality of the surface produced. This surface is highly resistant to abrasion, freezing and thawing and also possesses all the other desirable qualities that good concrete should have. Sticking of the form liner to the concrete surface is the greatest obstacle to overcome.

INTRODUCTION

Engineers have been giving considerable thought to the production of concrete more durable under exposure to weathering. Some of our earlier works built with dry, hand-tamped concrete are standing up better than some structures of a later date. From 1910 to 1930 was the period of wet concrete. Until about 1920 the true significance of the effect of overly wet concrete on its durability was not fully appreciated by most engineers. Abrams' work on water-cement ratio focused attention on the action of the water in the concrete and its effect on strength, wear, and other desirable qualities. Engineers began designing concrete for strength. Obtaining necessary strengths, some of them neglected the quality of durability. In other words, 1500- or 2000-lb. concrete was not always durable when exposed to the weather. Many dams were built with 1500-lb. concrete. Even 20 to 25 years ago, before the introduction of higher strength cements,

*Received by the Institute Feb. 28, 1941.
†Materials Engineer, Kentucky Dam, Tennessee Valley Authority.

concrete of this type could be obtained with nine-gallons of water per sack of cement. Today, we do not consider a nine-gallon concrete durable even when it goes 3000 p.s.i. at 28 days.

With the introduction of vibration in the late twenties, the dry concrete of 40 years ago came back into its own. Here was an instrument for placing concrete which enabled us to reduce the water-cement ratio, eliminate segregation and laitance due to overwet mixes; at the same time reduce the cement content and still obtain an increase in the strength and other desirable qualities. Vibration, however, has not eliminated all of our troubles, in spite of the use of dry mixes. We are still troubled with free water coming to the surface and increasing the water-cement ratio of the concrete where we least desire it. The use of absorbent forms, the writer believes, will eliminate most of this trouble.

Absorptive forms are not a new development. Perhaps the most notable work along this line was done by John J. Earley. Mr. Earley's contributions to A. C. I. literature in more than 20 years have been numerous and valuable.* Engineers have been slow to recognize how his work with absorptive forms along architectural lines could be applied to the less ornamental and more massive structures with which they deal. Failure to take advantage of the absorption factor in the production of high quality concrete has been due to the lack of an economical medium, or substance, with which to withdraw excess water.

LABORATORY TESTS

As a result of work by the Bureau of Reclamation at Denver on the testing and use of absorbent linings for concrete forms,† tests were started by the concrete laboratory at Kentucky Dam to furnish further information.

The Bureau tests indicated that considerable improvements could be made, both in the appearance and quality of concrete surfaces placed against vertical and sloping forms, by lining the forms with highly absorptive wall boards about one-half inch thick, made of ground cane, wood pulp, or similar materials. When concrete was placed against forms of this type, practically all pitting and voids so often found on sloping and vertical concrete surfaces were eliminated. The tests also showed that the quality of the concrete at the surface had been improved considerably as tested by means of freezing and thawing. As stated in the Bureau report, the concrete had a "case-hardened" surface.

*See A. C. I. Directory, 1940-41, p. 40.
†*Engineering News Record*, Jan 4, 1940 and *Western Construction News*, Feb. 20, 1940.

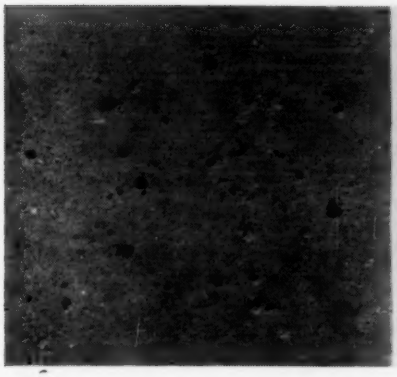

FIG. 1—A FULL FACE OF 10-IN. CONCRETE CUBE CAST AGAINST WOODEN FORM

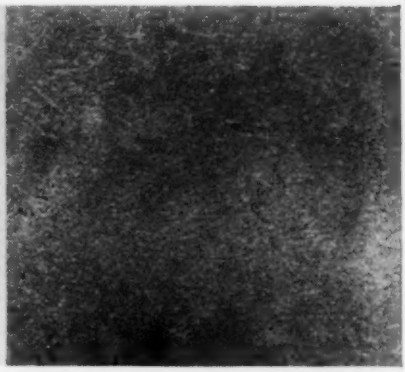

FIG. 2—A FULL FACE OF 10-IN. CONCRETE CUBE CAST AGAINST WALL BOARD FORM

The increased hardness and strength of the concrete at the surface is due to the greatly lowered water-cement ratio brought about by with-drawing the excess water at the surface by the absorbent form lining before hardening takes place. This is the reverse of the effect from oiled wooden forms. This is particularly true of a sloping surface. such as a dam spillway, where the free water collects just under the forms.

Preliminary tests were made at Kentucky Dam by placing concrete in 10-in. cube molds lined with various commercial products obtained from local materials dealers, similar to absorptive form linings since developed, but not having the high absorptive qualities of the materials now on the market. Several manufacturers have since begun producing an absorbent form lining which has the same appearance as their original products but having a much higher total capacity for absorption as well as a greatly increased rate of absorption. The preliminary tests showed practically the same results as obtained by the Bureau: that is, the elimination of voids and pits as well as a greatly increased density of the concrete at the surface, with the more highly absorbent materials giving the better results. Fig. 1 shows one side of a cube cast against an oiled wooden form, while Fig. 2 shows another side of the same cube which was cast against an absorptive form lining of comparatively low absorptive qualities.

As a result of these tests, requests were made of a number of manufacturers to furnish samples of materials to be used for absorptive form lining. Three manufacturers responded. The others had not yet developed materials to meet the specifications.

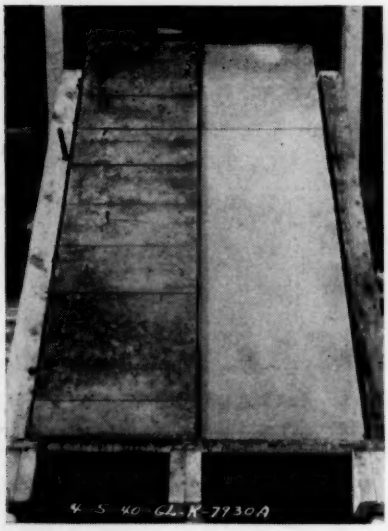

FIG. 3 — SLOPING FACE OF CONCRETE IMMEDIATELY AFTER REMOVAL OF FORM—OILED WOOD AT LEFT; ABSORPTIVE WALL BOARD AT RIGHT

To simulate placing conditions for the spillway section of a dam a wooden test form was built on a 45 degree slope. The form was 3 feet wide by 5 feet 6 inches long by 8 inches thick. One half of the under side of the top panel was lined with a section of absorptive form liner, while the other half was painted with form oil. Concrete was placed in the form in four lifts and compacted by vibration. At the end of 24 hours the top panel was removed. The form lining generally came off without sticking. If, however, the form was left on longer than 24 hours the sticking was rather pronounced and the lining had to be scraped off the concrete with trowels or putty knives.

None of the absorbent materials could be used a second time. Fig. 3 shows a slab immediately after removing the top panel. The right half of the slab was in contact with the absorptive lining.

Another interesting test showing the relative porosity of the concrete was made by throwing water against the face of the concrete. The water penetrated the face produced by the oiled wood causing a slight bubbling effect and the concrete surface was dry in about five minutes. On the other hand, the surface produced by the absorbent form lining remained wet for a considerably longer period, most of the water eventually evaporating. None of the bubbling effect observed on the concrete produced by the wood form was observed on the concrete produced by the absorbent form. If, on the other hand, the concrete was soaked over night and then allowed to dry, the face cast against the absorbent form lining dried first. This was due to the greater amount of water absorbed into the concrete cast

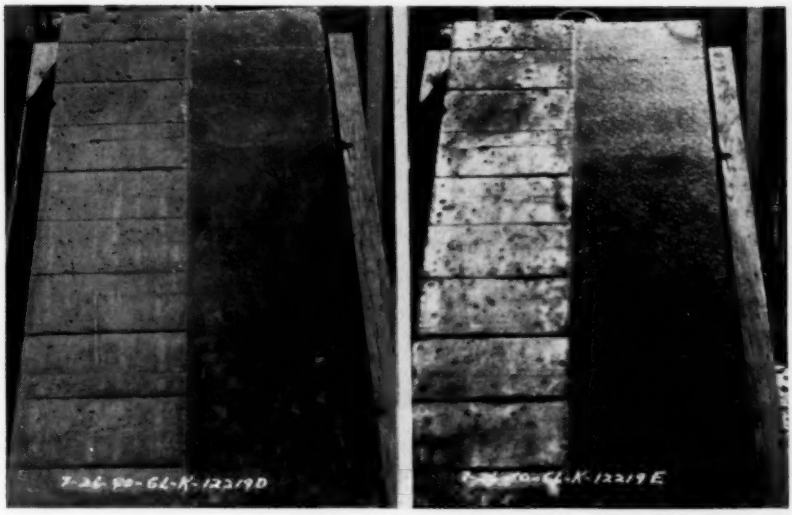

FIG. 4 (LEFT)—FACE OF CONCRETE SLAB IMMEDIATELY AFTER WETTING

FIG. 5 (RIGHT)—FACE OF CONCRETE SLAB TEN MINUTES AFTER WETTING

against the wooden side of the form. Fig. 4 shows a slab just after throwing a bucket of water against the face of the concrete. Fig. 5 shows the same slab about ten minutes later. Note the drying effect on the left hand side of the slab where the face of the concrete came in contact with oiled wood form. This test was made when the concrete was about two days old. The same relative effects can still be observed, although to a lesser degree, after six months exposure to the weather.

After the test slabs had been moist cured for seven days and then allowed to cure in the outside air until 28 days old, the lower half of several of the slabs were sand blasted. Each side was given an equal amount of sand blasting. Fig. 6 is a close up view of a section of one of the slabs after this treatment. Note the exposed coarse aggregate on the left hand side where the concrete was placed against the wooden form, whereas the concrete on the right, which was placed against an absorbent form, shows practically no wear with the exception of a few voids. The voids showing in the narrow vertical section at the center of the picture are due to the concrete being placed against a half inch wooden strip which was used to protect the edge of the absorbent form lining when concrete was placed against it. On all slabs, the side on which the absorbent form lining was used showed

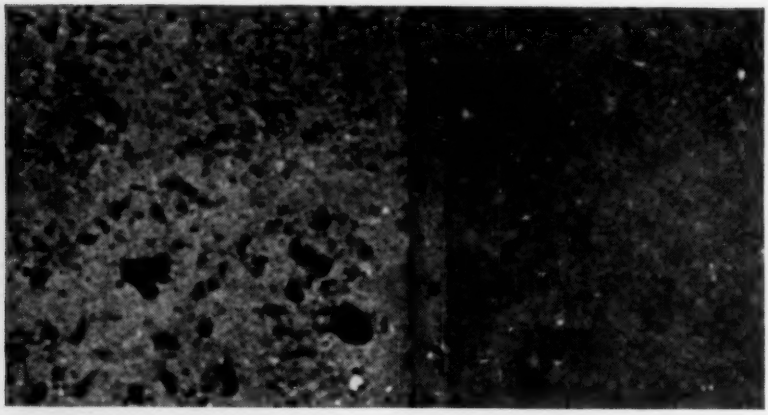

FIG. 6—SECTION OF CONCRETE SLAB AFTER SAND BLASTING (ABOUT THE SAME AREA OF SURFACE AS IN FIG. 1 AND 2). OILED WOOD FORM AT LEFT; ABSORPTIVE WALL BOARD FORM AT RIGHT

considerably less wear after sand blasting than did the side on which the oiled wood sheathing was used. Slabs using the most highly absorptive material showed the least wear.

Water thrown against the surface indicated that the concrete placed against the various absorptive form liners was denser at the face than the same concrete placed against oiled wood forms. To confirm this, tests were made on 6 in cubes cast in absorbent molds and in wood molds of the same size. The data in Table 1, plotted in Fig. 7, 8, and 9 show the results of these tests. Eight specimens were made with each type of form lining. Four of the eight cubes were cured in the moist room, while the other four were left in the air of the laboratory until tested at 28 days. The specimens were weighed upon their removal from the forms, 24 hours after molding, as well as at intervals during the curing period. In every case the specimens cast in absorbent molds and air cured lost less weight than similar specimens cast in wood forms. It might be argued that the reason for this was that part of the water was withdrawn from the concrete by the absorbent molds before the specimens where removed from the forms and that the weight loss is not an indication of the density of the surface concrete. If this were true, the specimens cast in the absorbent molds and cured in the moist room should have gained more weight than those cast in oiled wood forms and cured in the same manner. This was not the case however, as the cubes cast in absorbent forms gained less weight than those cast in wood forms, which indicates that the

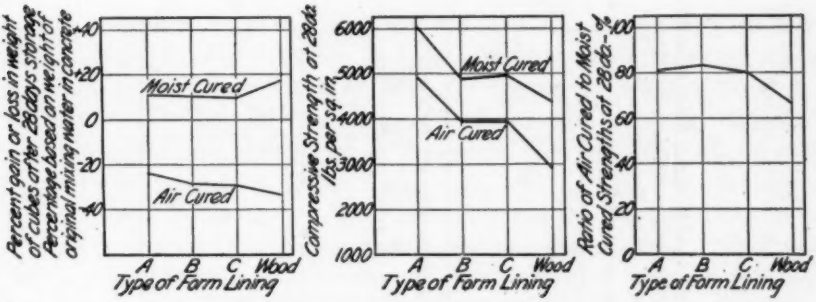

FIG. 7, 8 AND 9—COMPARISON OF 6-IN. CONCRETE CUBES CAST IN ABSORBENT MOLDS WITH CUBES CAST IN OILED WOOD MOLDS. FIG. 7 (LEFT) SHOWS CHANGE IN WEGIHT OF CONCRETE; FIG. 8 (CENTER) COMPRESSIVE STRENGTH; FIG. 9 (RIGHT) COMPRESSIVE STRENGTH RATIOS

TABLE 1—EFFECT OF CURING ON SIX INCH CONCRETE CUBES CAST IN MOLDS OF ABSORBENT FORN LINING

Strength, weight, and curing tests of 6-inch concrete cubes.
Aggregate: Tennessee River Sand graded from 0-No. 4 and Star Quarry Limestone graded from No. 4-1½ inches.
Type "B" Portland Cement.
Mix: 1-2.5-4.5 by weight: Water-cement ratio .60 by weight: Slump 3 to 4 inches.
Specimens cured as indicated. Weighed at 1-7-14-27 and 28 days.
All specimens tested wet: Air cured specimens soaked in water for 24 hours before testing. Each value is the average of four tests.

Type of Form Lining	Method of Curing	Change in Weight After 27 Days Curing—Per Cent of Mixing Water		Comp. Str. at 28 Days—p.s.i.	Ratio of Air Cured Str. to Moist Cured Str.—Per Cent
		Gain	Loss		
Oiled Wood.....	Moist Room	+16.4		4410	
	Air of Lab.		−34.1	2910	66
Ab. Lining "A".....	Moist Room	+10.1		6040	
	Air of Lab.		−24.4	4900	81
Ab. Lining "B".....	Moist Room	+ 9.8		4830	
	Air of Lab.		−28.8	3960	82
Ab. Lining "C".....	Moist Room	+ 9.1		4930	
	Air of Lab.		−29.5	3940	80

absorptive liners did produce less absorptive concrete. The results of the weight tests are plotted in Fig. 7.

In all tests the cubes cast in absorbent molds gave higher strengths, both for air and moist curing, than did cubes cast in oiled wood molds. As the absorptive effect of the mold did not penetrate through the entire specimen and merely lowered the water-cement ratio at the face, the actual strength of the concrete at the face was probably considerably more than indicated by the tests. The strengths are plotted in Fig. 8.

One of the most significant features brought out by the tests was the relation between the strengths of the moist cured cubes and the air cured cubes made with the same type of form lining. All three

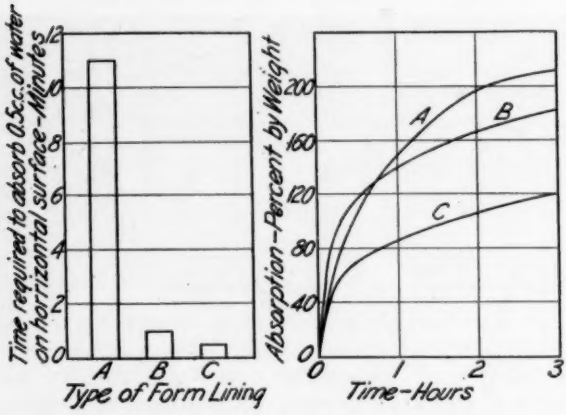

FIG. 10 (LEFT)—SURFACE ABSORPTION

0.5 cc of water placed on top side of 4 x in. piece of absorbent lining. Time shown is period required for water to absorb completely into test piece.

FIG. 11 (RIGHT)—ABSORPTION BY IMMERSION

12 x 12 in. pieces of absorbent form lining immersed in water—surface dried and weighed at intervals shown.

groups of specimens cast in absorptive molds gave air cured strengths equal to 80 to 82 per cent of their moist cured strength, whereas, cubes cast in oiled wood molds and cured in the air were only 66 percent as strong as their companion specimens which were moist cured. All specimens were tested wet, the air cured specimens being soaked in water 24 hours before testing. The strength ratios are plotted in Fig. 9. The higher strength ratios obtained from air cured specimens cast in absorbent molds again indicates the great density of the concrete at the face, which allowed less water to escape from the concrete after setting, and thereby provided more for hydration during the hardening process. Most of the surface curing agents now sold are based on the same principle: That is, the retention of the free water in the concrete to do the curing. The writer does not believe, however, that curing should be dispensed with when concrete is cast against absorptive molds, except in cases where staining or discoloration of the surface is objectionable.

Fig. 10 and 11 show the results of absorption tests on the three types of form lining. The test results shown in Fig. 10 were obtained by placing 0.5 cc of water on the top side of a small piece of room dry material and noting the time required for complete absorption. The tests tabulated in Fig. 11 were made by completely immersing 12 12-inch specimens of the material in water for the time intervals shown, surface drying, and then weighing. A comparison of the two

figures indicates that the material showing the most rapid surface absorption did not necessarily give the greatest total absorption when a specimen was completely immersed. Earlier tests made on commercial boards not sold for this purpose, indicated no surface absorption was obtained after testing for six to eight hours, but showed absorptions as high as 50 per cent by weight when the specimens were completely immersed in water. Concrete placed against boards of this type showed surfaces practically free from voids. After the usual sand blasting treatment however, a number of voids were uncovered just under the surface skin. In general it was found that the more highly absorptive materials gave concrete that was practically free from voids even after the sand blasting treatment.

Freezing and thawing tests on four inch concrete cubes molded in absorbent forms as well as in oiled wood forms are now under way. The results are not yet available.

FIELD OBSERVATIONS

Over 200,000 sq. ft. of absorptive form lining have been used at Kentucky Dam to date. It has been used on all exposed surfaces in the lock. Exposed surfaces are those starting from a point several feet below low water elevation, which will be exposed to weathering action. No attempt has been made to use it on horizontal surfaces as tests indicate that its use here is not practicable as yet. Pressure against the liner by the concrete is needed to achieve the best results, and no practicable method has yet been devised to exert this pressure as well as to give the desired grade and finish to a horizontal surface. The material will also be used for all exposed sections of the dam and power house.

No particular difficulties have been encountered in applying the material to the forms. The liner is received from the manufacturer in 5 by 8 foot sheets, in packages of six sheets. It is stored in small wooden sheds 2 ft. by 10 ft. by 6 ft. in size at various points on the job. Each shed is large enough to house the material required for lining a single lift for any particular pour. When a form is ready for lining, the shed is hoisted into the block by means of a crane. By using the method described, the liner is protected from the weather until ready for use. If a rain comes up during the nailing to the form, or while concrete is being placed, the lining is protected by means of tarpaulins hung from the forms. As an added precaution the liner is not nailed in place until the form is ready to receive concrete.

The liner is nailed to the form with four-penny common nails. The nails are spaced approximately six-inches apart along the edges and

twelve inches apart in the center of the panel. They are driven flush with the surface. Driving too deep will cause small bumps on the concrete surface.

No changes in the consistency or workability of the concrete from that used with ordinary wooden forms are required. Care must be taken to keep vibrators away from the face of the form as the material is damaged if struck by a heavy vibrator. This feature has caused no trouble thus far, and at the same time, no honeycomb has occurred when the liner has been used. The concrete surfaces are free from practically all voids or air pockets found on all concrete placed against non-absorbent forms. The form liners have also eliminated all ragged horizontal joints caused by grout from the form above running down over the old concrete face of the lift below. It has also eliminated ragged surfaces caused by a slight spring or "give" in the forms during placing operations.

In stripping, forms should be removed as quickly as possible to prevent the absorbent liner from sticking to the surface. If left on longer than 24 hours this sticking, in some cases, is rather pronounced; less pronounced in cool weather. If stuck to the surface, the material can be removed with a stiff wire brush. Wetting the surface helps considerably in removing the adhering material.

Sticking to the surface and inability to re-use are the greatest faults of the material. Even this however, cannot be considered serious when its advantages are considered.

Concrete placed against absorptive lining has a pleasing texture. This texture, of course, varies with the type of material used. The surface is free from pits and voids and no finishing of any kind is required after the forms are removed. If the concrete is vibrated too close to the face, occasional bumps are caused by the large size aggregates (3 and 6 in.) pressing against the liner. The surfaces are free from all hair checking or crazing as well as sand or water streaking sometimes found on vertical or sloping faces placed against non-absorbent forms.

If panels of standard size are being used the manufacturer should be asked to furnish the absorbent form liner in sizes to fit the panels with the least waste in cutting. If there is a difference in absorption between the two faces of the lining material, one side should be marked so that the most highly absorbent side can always be placed against the concrete.

SUMMARY AND CONCLUSIONS

Following is a summary of the results and conclusions of laboratory tests and field observations on the use of absorptive lining for concrete forms:

1. Practically all voids and pits, always found on formed vertical and sloping concrete surfaces, are eliminated.
2. The surface of concrete placed against absorptive forms is considerably stronger, more resistant to wear and erosion, and has greater impermeability than concrete placed against non-absorbent forms.
3. Lack of curing does not reduce the strength of the concrete to the same extent as concrete placed against non-absorbent forms.
4. Absorbent forms cannot be used more than once.
5. Until the manufacturers perfect a material that will not adhere to the concrete, the forms should be removed within 24 hours to prevent too much sticking to the concrete face.
6. Due to the fact that pressure of the fresh concrete is required against the lining to produce the best results, its use against horizontal top surfaces is not recommended until a suitable method for doing this has been found.
7. In view of the greatly improved quality of concrete surfaces obtained, the small additional cost per cubic yard of concrete should not be given too much weight. It is also highly probable that compensating factors will more than offset the additional cost. Among these factors are the use of cheaper lumber for lagging, the saving in labor of applying form oil and in the cleaning of the panels between pours. It is also highly probable that by increasing the water-cement ratio and reducing the cement content the surfaces produced will still be considerably better than we are obtaining today with lower water ratios and higher cement contents. Concrete strengths obtained in most structures built today are considerably higher than required to carry the designed loads, but concrete of this quality is required for durability. It, therefore, is quite logical to assume that if durable surfaces can be produced to resist the weathering action, the main body of the concrete will still have plenty of strength to carry the load. At the same time the lower cement content will aid materially in reducing cracking, and saving in cement will offset the additional cost of absorbent form lining.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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Construction of the Terminal Building—Washington National Airport*

By W. E. Reynolds†

SYNOPSIS

Records briefly the nature of the project and the practice in form work—materials and assembly, the concrete mix, placing and finishing methods employed in obtaining satisfactory results in the construction of the Terminal Building, Washington National Airport.

When we began designing the terminal building, Washington National Airport, it was necessary to bear in mind several rather staggering facts. First, government owned or designed buildings must always stand for the best in a national architecture, as well as meet functional requirements; second, the Washington National Airport, at the Nation's Capital, must be an outstanding example of efficiency in design; third, definite amounts of space were required for the accommodation of specific numbers of activities which must be housed; and, fourth, all of these had to be supplied by the expenditure of a fixed sum of money, for, although a large amount of money was appropriated for the new airport, the major portion was used in making a magnificent landing field from a large area of bog.

Placement of the presently designed buildings on the site must allow for expansion of the project. The original contract covered only one hangar, which obviously provided too little hangar space for the use of the three airlines now serving the Capital, and the Army, Navy, and Civil Aeronautics Administration planes which may need housing on the field. Since that original contract was let, an appropriation has been made for the construction of five additional hangars,

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†Commissioner of Public Buildings, Public Buildings Administration, Washington, D. C.

FIG. 1—AIR VIEW OF CONSTRUCTION OPERATIONS ON NEW AIRPORT

now in process of design, and it seems more than likely that still further hangar and garage space will be required.

The project is being financed largely from funds of the Public Works Administration and the Work Projects Administration. Its construction is being carried forward under the direction of the Civil Aeronautics Administration, working through various government agencies, such as the Corps of Engineers, the Public Roads Administration, and the Public Buildings Administration. These agencies in turn cooperate with the National Park Service, the National Capital Park and Planning Commission, and the Fine Arts Commission in the development of the project. For purposes of administration, the Civil Aeronautics Administration appointed an Interdepartmental Engineering Commission for advice and counsel. This Commission has done an exemplary job.

The design and supervision of construction of terminal building and hangars, together with the planning of approach roads to these structures, were assigned to the Public Buildings Administration. Due to the proximity of the hangars to the Mount Vernon Memorial Highway, these structures in themselves were required to have real architectural merit. Because of the large door opening requirements, and low height limitations, the superstructure is structural steel. The foundations, floors, sides and rear of the hangars are in reinforced concrete.

Great study was directed toward the design of the Terminal building as a serious mistake here could not be condoned. Available data

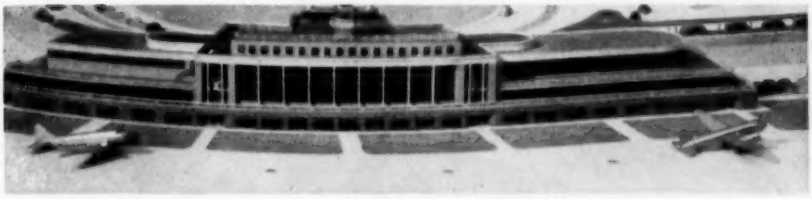

FIG. 2—PHOTOGRAPHED FROM MODEL OF TERMINAL BUILDING, WASHINGTON NATIONAL AIRPORT

on comparable airports throughout the world were studied. An engineer was sent to Europe to examine at first hand the actual operation of the larger airports. The commercial airlines serving Washington, placed at the Government's disposal their most able technicians. The technical staff of the Civil Aeronautics Administration rendered valuable assistance. The Consulting Board of Architects and the Consulting Board of Engineers, on regular assignment with the Public Buildings Administration, followed closely the development of the plan. Finally, the architectural conception of the building met the critical review of the Fine Arts Commission. Surely, enough talent was employed to secure a satisfactory structure.

With this general background this paper is concerned with details of construction, rather than the planning of the structure itself. With that thought in mind, the following observations may be of interest.

As designed, and almost completed, the Terminal Building is a three story, combination reinforced concrete and structural steel building, designed in accordance with the provisions of A. C. I. and A. I. S. C. Building Codes. The building is 540 ft. long and 90 ft. wide, with the center portion and one end having a radius of 412 ft. It rests on a foundation of cast-in-place concrete piles, figured for a maximum load of 30 tons per pile.

The outstanding structural features of the building are:

(1) Reinforced concrete cantilever balcony in the main waiting room, with a 20 ft. 6 in. overhang and a length of about 180 ft.; (2) second floor and high roof are reinforced concrete slab construction without drop panels or capitals. This was necessary to obtain a flat ceiling with the minimum of depth in floor construction; (3) the third floor construction over the main waiting room is of the cellular steel type, supported on built-up steel girders 4 ft. deep, with a span of 57 ft. The cellular steel type was used to provide adequate electrical distribution. To permit maximum window area

on the field side of the waiting room, the girders were supported at that end by steel pipe columns filled with concrete; and, to avoid excessive bending of these columns due to girder deflections, the girders bear on rocker arms which rest on top of the round columns.

FORMS

The usual custom in this locality, when building forms, is to use studs 12 x 16 in. on center; but, on the west coast where the best concrete structures have been erected, experience has shown that forms built with studs that wide permit a cupping of the sheathing. To eliminate that cupping, it is necessary to move the studs closer together. On this project the contractor was allowed to use studs 12 in. on center if the plywood were ¾ in. thick, or 8 in. on center if the plywood were ⅝ in. thick. He chose to use ⅝-in. plywood and studs 8 in. on center, which would permit a general sharpening of the lines of the form work. The wales are 24 in. on centers, and all studs and wales are made of 2 x 4 in. stock to allow flexibility in alignment. The corners were firmly braced to prevent slippage of the formwork.

In looking at a new structure, the eye immediately picks up the lines; and, if they are true, minor imperfections escape notice. If the lines are not true, then the eye goes on critically to pick out every imperfection in the structure—hence the value of perfect alignment.

It was found that new and used material, either lacquered or oiled, gave different textures to the finished surface, depending on the absorptive properties of the material. New lacquered plywood produced a hard, non-absorbent, compact surface texture; while new, oiled material showed considerable absorption and gave splotches of a darker color, on account of the more porous texture of the form surface. Oil has a tendency to dry before the concrete is placed, and does not produce a non-absorptive form surface. The sample submitted for approval should show the desired texture of the finished surface, and, as the texture affects the tone of the color, no change in form lining material can be permitted from that used in the approved sample.

After experimentation, it was found that plywood forms treated with one light application of special form lacquer produced satisfactory results. The re-use of the forms was permitted once, with light oiling. A second coat of lacquer is most unsatisfactory, as the second application tends to melt into the first coat, and both remain tacky. However, on any given wall area, the forms had to be made of either all new pieces, or all re-used pieces to get uniform results in texture and appearance. Having been used twice for architectural concrete,

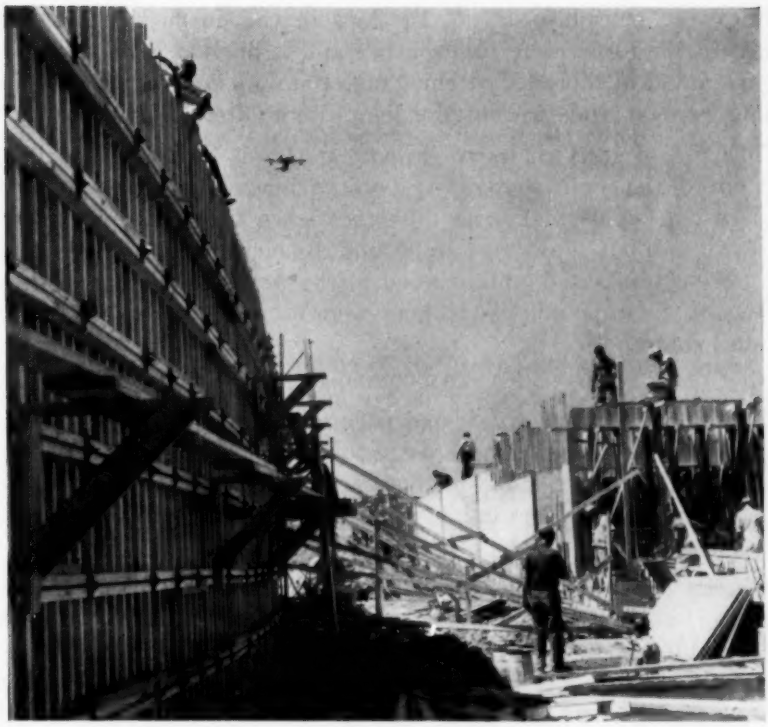

FIG. 3—CURVED FORMATION OF TERMINAL BUILDING ON LOWER DRIVEWAY

the forms could not again be used for that purpose, but they could be, and were used many times for the structural concrete slabs, unexposed walls, etc.

On curved surfaces, the sill was cut to pattern, with the studs set to this line and then sheathed. The panels for exposed concrete ceilings were all laid out symmetrically rather than at random, which gave a very satisfactory effect.

Waste molds were used for cornices, and also for models. These waste molds were made locally, and erected in the forms and bracing. Studs and wales were made continuous across the molds to produce continuity of alignment. It was found that wales, or studs that were broken or spliced, or offset at the joints between waste molds and wood forms adjacent, allowed the lines to break and wave.

These methods of form work were successful in preventing board marks of the backing from showing in the finish. Nails in the form

lining were placed so as not to show in the finished surfaces; the joints in the forms were sufficiently tight to prevent bleeding; cracks were pointed with plaster patching material. All vertical joints of form lining were on studs and interior joints were mitered.

The use of steel chairs to support steel was discontinued, as the bottoms of the chairs were exposed on the finished face of the concrete, and rusted. This makes no difference when the surface is interior, and painted, but shows rust stains and discoloration when the surface is exterior, unpainted, and exposed to the weather. Precast concrete blocks of the same mix as the pour were used to support steel in lieu of steel chairs.

THE CONCRETE

The items that are considered necessary to architectural concrete are: (1) An excess of cement paste to insure complete filling of form; (2) Admixture, when sufficient fines are not present to give increased plasticity; (3) Proper grading of fine aggregates to obtain the necessary fines; (4) Absolute control of the water.

Class A concrete, with 5.2 sacks of bulk cement per cubic yard, was specified; and, at the request of the construction engineer, this was increased to 6.0 sacks per yard to produce the necessary excess cement paste. This job was fortunate in having both the aggregate and batching plants on the site. This allowed absolute control of the washing and grading of the aggregate, and produced the necessary fines. Being obtained as a residue from the dredging operations, the sand was a bit dirty, but was not washed too thoroughly for fear that all desirable fines would be lost. The gravel was crushed when necessary to reach the required grading with a maximum size of one inch.

The moisture content of the aggregate was checked at the stock piles, and water was measured and placed in the mixing trucks at the batching plant. This gave absolute cement-water ratio control for all concrete.

Having been measured by weight at the batching plant, the sand, gravel and water were placed in mixer trucks of 3.5 cu. yd. capacity, and the cement was added from another chute located a short distance from the batcher plant. The concrete, delivered in the transit mix trucks, produced a uniformly satisfactory mix. A five inch slump was allowable, on account of the nice workability and placticity of the mix.

The concrete was handled with buggies, which practically eliminated segregation of materials; and, in combination with the use of tremies,

also prevented "spatter" on forms above the pour which, if it dries before setting, leaves the surface pitted. The concrete generally was vibrated. Both gasoline and electric machines were used, and the electric type was found superior to the gasoline because the motor unit was less cumbersome and workmen could not cut the speed and vibrate too slowly. In some locations there was not enough clearance to use vibrators, and the forms were malletted from the outside by *experienced* workmen, so that the results obtained equalled those obtained with vibrators. Hose lines were kept available to wash the seepage of the pour away, and prevent staining the work previously poured below.

The wall ties used were designed to incorporate spreader action by nailing the rods to the studs on each side. Holes are provided in these rods for this purpose; but the device did not prove successful, as it is not possible to control the spacing of studs so that they occur close enough to the wall ties to be nailed to. Unless ties are nailed in contact with studs, the spring of the nail allows variation in the thickness of the wall. Precast concrete spreaders incorporated with the ties will give positive spreader action. These spreaders, of course, have to match the color of the wall.

The concrete was cured at least six days—although in some cases it was necessary to remove the forms in four days, in which case the contractor covered the concrete with burlap, and kept it wet for the remainder of the required curing period. The tie rod holes were grouted immediately upon stripping, in order to secure proper bond, using a paste of 20 per cent white cement 30 per cent gray cement and 5 per cent sand.

The mix for the wash was determined experimentally on the sample to ascertain what mix and color would even out the variations in colors and patching. It was found that the wash must be the same color as the poured concrete, since the applied finish, or wash, does not cover or mask the poured concrete enough to allow for any deviation from the basic color of the material poured. Lighter or darker colors fill up board marks and rough surfaces only, and show undesirable contrast. The wash was applied by means of burlap, the wall surface wetted before application of the wash which was of paste consistency.

The wall was given a coating of a mixture composed of 20 per cent white portland cement, 30 per cent standard portland cement, and 50 per cent sand passing 80 mesh. This coating was wiped on to a thickness of $\frac{1}{8}$ in. Thirty minutes later the coating was scraped off

with a steel trowel, then rubbed with burlap. No rubbing with carborundum stone was done, except in smoothing out rough spots at patches, which amounted to practically nothing considering the size of the job.

There are one or two outstanding features of the terminal building that I should like to mention: It was found that by making the joints at the top of the rustication, two very desirable effects were obtained. First, the rustication was made sharper, and then the joints were more easily and effectively concealed. Another feature is the lettering on this building, which compares very favorably with any of the carved stone lettering to be found in the city. The letters are sharp and clear and very legible at considerable distance from the building.

It is believed that the success in obtaining satisfactory lettering was due to the design of the letters which are 18 in. high, the quality of the forms, the careful placement of concrete and stripping of forms. The typical cross section of the incised letters is $1\frac{1}{4}$ in. wide, $1\frac{3}{8}$ in. deep, with the first $\frac{3}{8}$ in. at right angles to the surface and the balance forming a "V." The sharpness of lines is due to the full right angle corners of the letters at the surface of the concrete. Forms for letters were specially made of clean white pine, carefully fastened to form linings and swelled by soaking in water just before installing. The form letters were not removed from the concrete when the forms were stripped, but allowed to shrink and loosen of their own accord when dried out. This prevented spalling and damage to the sharp corners.

In short, we are quite delighted with the results obtained through the use of architectural concrete in this instance. This medium of construction permits long flowing lines and curved surfaces so suitable for a building serving our newest form of transport. It also reflects the feeling that a building servicing such a rapidly changing industry should have good taste without extravagance.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

Vol. 37 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by W. E. Reynolds:

Construction of the Terminal Building—Washington National Airport*

BY GEORGE SHELDON, A. J. BOASE, D. E. PARSONS
CHARLES MACKLIN AND THE AUTHOR

CONVENTION DISCUSSION

A. J. Boase—I understand there was an especially interesting problem in placing the columns. Will Mr. Sheldon describe that?

George Sheldon—Having anticipated that question, I have some comments written out here which I will read:

The architectural concrete columns for the loggia of the Terminal Building (at the Washington National Airport) are two feet square, and 25 feet high, enclosing a 10-in. structural steel column. Built into each column is a continuous flush lighting fixture, 8 in. wide, extending from 8 ft. above the floor to 1 ft. below the top of the column to provide indirect illumination for the loggia. Each fixture had internal bracing to resist the hydrostatic pressure of the concrete during the pouring process. The reinforcement of the columns was of typical fireproofing design, composed of hoops and vertical rods. The forms were worked from $\frac{5}{8}$-in. plywood such as is used for ping-pong table tops and which is available in 5 x 9-ft. sheets. Horizontal joints were placed at the third points. The plywood was backed up vertically at the corners and third points with 2 x 4-in. material laid flat. The corners of the columns were not chamfered, but particular care was taken to hold the plywood together firmly to prevent any leakage. The column clamps were spaced from 4 in. at the bottom to 10 in. at the top. After lining up the forms, they were braced to the wall of the building,

*ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 633. The paper was presented to the 37th Annual Convention by George Sheldon, Construction Engineer, Public Buildings Administration, Washington.

and also to each other. On account of the difficulty of placing the concrete in columns 25-ft. high, and with structural steel and electric fixture obstructions, the mix was changed by omitting one-half of the gravel used in regular architectural concrete mix. The slump did not exceed six inches. The concrete was poured at the rate of 6 ft. per hour, and puddled with rods and also with agitation of the reinforcement from above. The forms were thoroughly malletted on the outside as the pour progressed. No tremies were used, as the rate of pour and the moist atmospheric condition prevented spatter from setting up inside the forms ahead of the pour. The richness and workability of the mix, as well as the thoroughness of puddling and mallett-ting prevented segregation of the aggregates. The results were highly satisfactory for the reason that there was no honeycombing, no leakage at external angles, no sand streaks, and less than the average number of air holes.

*D. E. Parsons**—I would like to ask how the cement and sand grout was applied and how long it was left in place before it was scraped off?

Mr. Sheldon—The cement-sand grout has several purposes. One is to fill air holes; the other is to clean down the concrete, and a third is to leave a slight meal on the surface of the concrete which will continue to effloresce in sunshine and rain. It was applied with either a wood float, a cork float or burlap. It was put on and left about 20 to 30 minutes and an area perhaps 50 feet in length would be covered in that period. That period allows shrinkage to take place; then the edge of a steel trowel was used to take it off. It requires a sharp edge to remove it. Then it is finally rubbed with burlap.

BY CHARLES MACKLIN†

Since the Terminal Building was given so much thought by so many government agencies, and the construction was so carefully supervised, it is only natural to assume that a very unusual job of concrete work was accomplished. Other projects will no doubt follow certain construction practices used on this project.

The use of $\frac{5}{8}$-in. plywood on studs 8 in. o. c. is unusual, and the question may be raised as to whether such rigidity in forms is necessary and whether it should be regarded as good practice. If a concrete pressure of 1000 lb. per sq. ft. were anticipated, the deflections in the end spans of a sheet of plywood would be only 1/100 in. In view of the fact that the 2 x 4 studs will vary by plus or minus $\frac{1}{32}$ in., it would seem that the lumber was a greater contributing factor to irregularities

*National Bureau of Standards, Washington.
†Normandy, Mo.

than the deflection of the plywood. If the stud spacing were increased to 12 in. o.c. the deflection would be 5/100 in., which is 1/240 of the span. If the concrete pressure were 670 lb. per sq. ft. the deflection would be 33/1000 which is 1/360 of the span. Recommendations on deflections for concrete work are often given as 1/270 of the span.

The horizontal shear in the plywood, studs, and wales are 80, 170, and 250 lb. per sq. in. respectively—computed for a pressure of 1000 lb. per sq. ft. This does not seem consistent with a balanced design.

It would be interesting to have a comparison in the alignment of these forms using studs 8 in. o.c. and a form using studs 12 in. o.c. to see if the additional labor and material created a wall which was superior.

AUTHOR'S CLOSURE

In connection with Mr. Macklin's discussion of the 8 in. spacing of the stud: It was deemed advisable to use 8 in. spacing to eliminate any noticeable deflection. The allowable deflection could probably have been noticed under certain lighting and inasmuch as the $\frac{5}{8}$ in. plywood was being cut into 12 in. strips to show horizontal jointing the placing of the studs on 8 in. centers was a safeguard to produce the desired degree of architectural perfection.

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The Grouting of Concrete Structures*

By T. C. Creaghan†

SYNOPSIS

This paper describes repair work done on the downstream side of certain hydraulic structures in Canada. It covers instances of stoppage of water flow back of the frost line, and repair of the structure in the dry. The temporary repair of surfaces so as to permit grouting of the interior under high pressures, and a brief description of the grout specification for different types of jobs.

The grouting of concrete structures is, of course, not new. One large concern in England has been continuously engaged in the grouting of structures since the beginning of the century and has agents over most of the world, specializes principally in pressure grouting mine shafts and tunnels, and also in the consolidation of the ground in the neighborhood of these shafts and tunnels. There have been many instances of grouting of the rock strata below existing dams. This has generally been done either from the downstream side, or from vertical holes through the dam into the rock. Present day specifications for dams require pressure grouting of the rock foundation during construction.

At the same time, pressure grouting has also been quite commonly used for the waterproofing of concrete, particularly in the final closing of the bleeds where the waterproofing is done from the off-pressure side. Grouting has also been used for the closing of leaks through both horizontal and vertical construction joints and fill planes. However, it is only quite recently that engineers have realized the value of grouting for the consolidation of weak or disintegrated concrete, thereby correcting an original shortage of cement and fine aggregate,

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†Creaghan & Archibald, Montreal, Que.

filling honeycomb, or replacing cement and fines leached from the concrete by the internal movement of water.

Between 1910 and 1925, there was a great boom in power plant construction in Canada. In some cases, to obtain sufficient money for these power projects, specifications had to be cut pretty close and mass-production methods used in the placing of concrete. In many instances, suitable aggregate could not be found economically close to the job, and water cement ratio control was not generally practiced. The consequence was that the concrete, in many of these structures, is not the best.

One thing that is common to most of these structures, is lack of sufficient cement; the second, is surplus of water—both of these resulted in a weak concrete with a great many voids. The third common cause of voids is lack of fines in the sand or gravel; and the fourth, though not so common, is use of dirty or seamy plums.

In recent years all these structures have required repairs, some minor and some of a much more serious nature.

In making these repairs, it is generally economically impractical to unwater the upstream side, so that most repairs have to be made on the downstream side. This, of course, entails the stoppage of the water passing through the structure, and unless this water is stopped back of the frost line, the permanence of the repair is greatly endangered. As the concrete of each job differs from the last, each brings a different problem of pressure grouting.

In 1937, a general repair was made on the main dam of a company at Shawinigan Falls—this dam was in two sections, and consisted of a series of piers on low foundations, connected by steel gates. (Fig. 1). The piers were badly disintegrated, particularly on the downstream side. There was considerable leakage around the gate guides, and these leaks were all stopped by pressure grouting before the repairs were made. Neat cement grout was used without sand or admixture. The repair work was then done in the dry with Gunite. To date, the leaks have not reappeared, except for one very small damp spot, which may or may not have been at one of the points grouted. The Gunite bond is not broken at this point, but there is slight cracking.

The repair specification for this and other similar jobs was roughly as follows: Faulty or disintegrated concrete was first removed with paving breakers and chipping hammers. The surface was thoroughly cleaned by sand, air, or water blasting, or a combination of the three. The surface was given a light coating of ground iron and salamoniac and left for 24 hours. This latter was to waterproof the old concrete to prevent it from sucking the water from the new Gunite, thereby

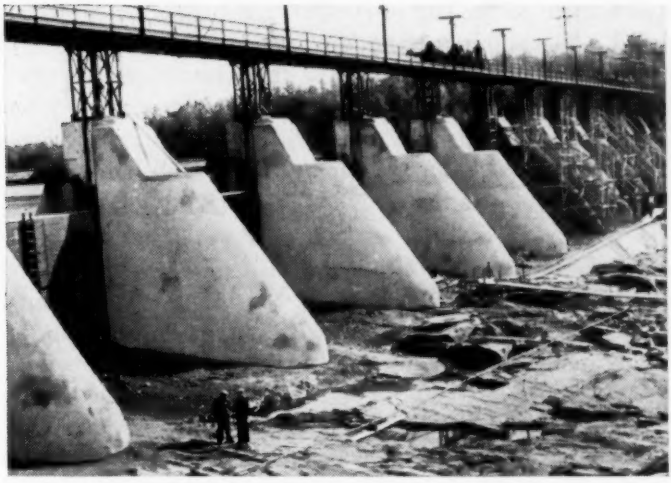

FIG. 1—DOWNSTREAM REPAIRS BEING MADE ON MAIN DAM AT SHAWINIGAN FALLS

weakening the bond. A suitable mesh was anchored to the surface with cinch anchors and bent nails. The Gunite was then applied in layers to bring the structure to its original contours. Care was always taken to water cure the Gunite for at least four days, and in some cases up to 28 days, depending on the thickness of the coat, the climate, weather conditions and exposure. On southern exposure in dry weather, curing was carried on for a full 28 days.

An example of the combined grouting of rock and concrete was the repair of one of the anchor piers of the spillway of a dam at St. Albans, Quebec, in 1936. The main spillway of this dam is a circular arch structure, with anchor piers at both ends. These anchor piers are stepped down according to the contour of the rock. The water was leaking quite badly through the rock, and also through the concrete of the anchor piers at one end. This water was first gathered into pipes, fastened into the rock and concrete with quick-setting cement and lead wool. These pipes varied in diameter from six inches down to one-and-one-half inches. The water channels in the rock and concrete were all connected, so that when the valves were closed in the bleeds in the rock, the pressure immediately increased in the bleeds in the pier. The water was leaking so freely through both rock and pier that the grout pressure had to be carefully balanced with the water pressure. The grouting was done alternately through different bleed pipes, but mainly through a six-inch pipe in the rock. Neat cement was used, mixed with fine sand and about one-half pound of

sodium carbonate to the bag of cement, until the main water channels were closed. The sodium carbonate was added to accelerate the set. Then fine cement was used to stop the small leaks. In all, about 3000 cubic feet of grout was pumped into the structure, probably one-third of which was lost on the upstream side. On chipping the piers to repair the surface, the grout was found to be well bonded to the old aggregate. This job is apparently satisfactory after three years.

Late in the season of 1939, a small portion of the back wall of the headgate house at the Shawinigan Falls plant, was grouted and repaired. This was a gravity section, consisting of what apparently had been good concrete, but full of derrick-size boulder plums. These plums were evidently not very well cleaned before placing. The structure was built in 1900. There had been seepage through this wall for a great many years, but the outside skin was hard and sound for thicknesses varying from half-inch to three inches, probably due to spading at the forms bringing the fines to the surface. This skin contained openings varying from palm size to several square feet, through which the water had been flowing since shortly after the structure was built. Behind this hard skin, the cement seemed to be washed out of the structure, and the concrete disintegrated for depths up to five feet. There were water channels around practically all of the boulders. There was a great deal of mud in the interstices. Later, diamond drilling showed that these interstices occurred throughout the structure. After removing the hard skin, the disintegrated concrete was cut out for depths varying from a few inches to five feet. Holes were then drilled into the water channels and through the boulders for depths up to eight feet. The mud was then blown out with alternate water and air blasting, and the wall grouted with various mixtures of common cement, reground cement, aluminous cement, ground iron, fine sand and certain lubricants and chemicals to increase or retard the set. It was found that generally, common cement and fine sand was best for this particular job. The grout came out in one place forty feet along the wall from the nearest grout hole. The section of grouted wall was completely dried, but, after the frost, showed damp spots along a T-shaped crack in the Gunite adjacent to an adjoining wall. This was apparently due to an error in joining the two walls with Gunite—separate movement of the two walls evidently cracked the "Gunite." The work was continued the next year by grouting through vertical holes diamond-drilled. The water was not completely stopped, but further test cores showed that the grout had filled the interstices, and was well bonded to the aggregate. This job is continuing.

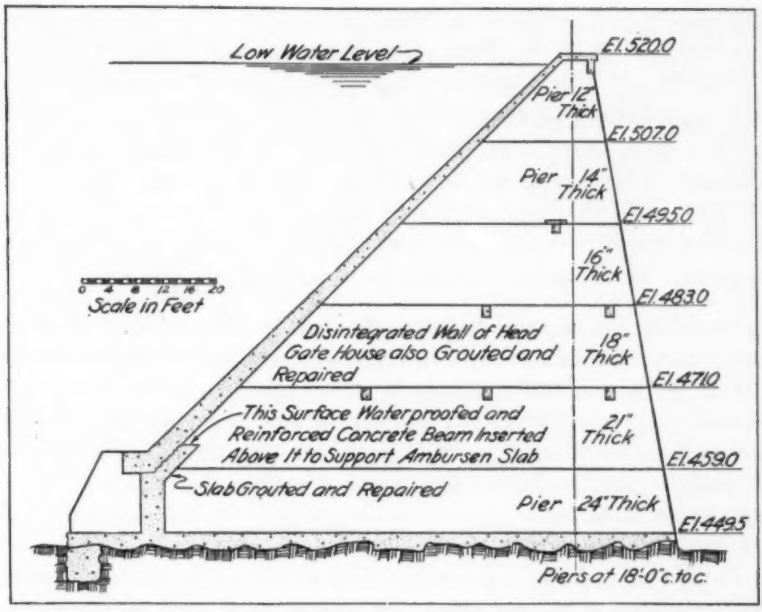

FIG. 2—CROSS SECTION SHOWING CLOSURE SLAB, ST. FEREOL, QUE.

A most interesting job was done on the Seven Falls dam near St. Féréol, Quebec. This is a 70 ft. head Ambursen dam, and the repair work had to be done at full head. The first of this work was done in the Fall of 1938, and consisted of the waterproofing and repairs of the headgate structure, the closure slab, and the penstock anchor pier.

Some years previously, during a washout, one of the supporting piers was washed out, causing the penstock to pull, thereby badly shattering the penstock anchor pier. Attempts had been made to stop the water by placing a heavy slab over the top. This slab was removed and the broken pieces of the pier tied back with heavy dowels. The pier was then thoroughly washed and grouted with cement, sand and a small amount of sodium carbonate. All leaks were stopped and have remained so, and the pier seems perfectly consolidated.

The closure slab was leaking so freely that the loss of water was a considerable factor. On examining this structure, it was found that, although the skin was hard over most of the slab, even the sand was washed out inside so that it was possible to work one's hand deep into the structure.

The top of the structure was first made waterproof, and a heavy reinforced concrete beam was placed on top to support the Ambursen

slab. (Fig. 2). The water was then gathered into bleeds and grouting commenced. This operation took about one month to finish, as the slab continually broke during grouting operations. Common cement, sand, and sodium carbonate were used for the grouting. This slab was so rotten, and the factor of safety of the Ambursen dam so well balanced, that the job was considered complete while the slab was still damp—further grouting might have again fractured the slab. The slab today is still damp and slightly cracked in places, but is apparently in the same condition as it was left two years ago.

The side walls of the headgate house, were the ordinary Ambursen piers thickened to about three feet six inches. The off-pressure surfaces of these walls were so bad that the grout pipes could be screwed into the wall without drilling. Further cutting on the wall was not advisable, as its strength was greatly impaired. As the wall was very thin and badly disintegrated on the downstream side, it was necessary to trap the grout to force it to travel laterally. To accomplish this, the wall was first covered with a thin coat of Gunite containing a high percentage of sodium carbonate. The whole wall was then grouted with a mixture of cement, fine sand and a lubricating admixture. In certain places, aluminous cement was used. After the wall was considered to be consolidated, the thin Gunite coat was removed and final grouting was done with aluminous cement. The wall was then cleaned and repaired with Gunite to its original contours. Only one very small leak has developed in two years in this wall. The owners consider this a highly successful job, and consider that this is the only practical method of repair, as it is impossible to get at the upstream side of this portion of the dam, and very difficult to reinforce it on the downstream side. The remainder of the dam, with the exception of the Ambursen slabs, was repaired during 1940, and the owners propose to repair the Ambursen slabs during the summer of 1941.

Another very interesting job was the dam at St. Raphael, Quebec. This is a gravity section dam filled within a few inches of the skin with man-sized stone taken from the excavation, and laid as in a masonry wall. These plums are of poor quality seamy rock. The downstream surface of this was grouted in two sections. When the second section was cut back, the interstices were found to be filled with grout forty to fifty feet from the nearest grout pipe. The bond between the grout and the old aggregate was found to be perfect. After the original water channels were stopped, a lubricating admixture was added to the grout to enable it to penetrate deep into the structure under low pressure.

Pressure grouting is not, by any means, a cure-all. In the case of a dam in Manitoba, where the concrete consisted of a very fine sand and a coarse aggregate, with a lean cement content, it was found that, although the structure would absorb water almost instantaneously, it remained wet for weeks in the dry Manitoba climate. The capillary attraction seemed so high that it was impossible to replace this water even with air. There was no apparent seepage through this structure, but the concrete was damp throughout and disintegrated rapidly under frost action.

It has often been found impossible to grout laitance seams, even with the addition of the finest admixtures. The water channels are sometimes so thin that the finest grout will not penetrate them, also the laitance will absorb water which grouting will not displace. For these reasons, the results of grouting operations were unsatisfactory at two large power plants in the Province of Quebec, and other methods had to be used. In one case the downstream side of the power house was housed and heated so that frost would not break off surface water-proofing. In the other, the upstream face was unwatered with floating caissons designed by the writer in cooperation with J. A. McCrory, Vice-President of the Shawinigan Engineering Co. The upstream face was then repaired and waterproofed with Gunite.

Sometimes it is found expedient to use grouting for the temporary stopping of water, so that repairs may be made in the dry. This method was used in certain piers at Bathurst, New Brunswick. In this case, it was necessary to cut completely around the stop-logs, leaving the end of the stop-logs cantilevered during the repair. Then temporary grouting was done, and the stop log slots repaired in the dry. The head was about thirty feet.

Three handicaps to date in grouting operations, have been the failure to get a large enough appropriation, the doubt as to results, and the fact that the grouting has to be done against high pressures.

Where an old structure contains a large percentage of interstices, due either to cement and a fine aggregate having been washed out, or an original lack of cement and fines, the conclusion is that the water should be stopped and the large interstices filled by a fairly quick-setting cement mixed with fine sand, and that, where this is possible, the further consolidation of the structure should be done with common or fine cement mixed with a stearate lubricant, and an admixture such as diatomaceous earth, fine clay, or powdered fuel ash. The whole to be mixed with fine sand and water to a consistency to suit the job. Where there is no water to fight, the problem is, of course, very much simplified.

As each job is a separate problem the success of the grouting is largely dependent on the skill and experience of the engineer and personnel performing the work. Not only do different jobs require different mixtures and consistency, but they may also require grout of rapid or slow set, and the most advantageous pressure may vary from a few pounds to a few hundred pounds.

Until recently, in the repair of dams, the object of grouting has generally been to stop the flow of water through the dam, thereby arresting further deterioration. The consolidation of the structure was always incidental and its value was only recently realized. In the case of the back wall of the headgate house of the Shawinigan plant, even though the flow of water is not yet completely arrested, the structure has undoubtedly been vastly strengthened by the binding of the aggregate with grout.

On structures where the water can be carried away with bleeds, or where leaks can be stopped at the source, the problem of consolidation by grouting is quite simple, and not disproportionately expensive. The grouting in this case should be done under low pressure, using very fine cement and admixture, and with the addition of a lubricating agent, such as calcium stearate, which will also retard the set.

However, as stated above, grouting is not always applicable. In the case of Ambursen type dams, as in the slabs at Seven Falls, the deterioration is mostly on the upstream face and grout forced into the structure from the downstream side would mostly find its way into the forebay, and would not be effective unless a coating could be placed on the upstream face to hold grout. The pressure necessary to force the grout through the dense concrete on the downstream side would be sufficient to force it clear through the more open concrete on the upstream side. It is for this reason that the writer could not recommend consolidation by grouting of a large storage dam recently repaired by the Quebec Streams Commission. This dam was unwatered and repaired with Gunite on the upstream side.

In summation, grouting aims at achieving three useful purposes: first, to stop leaking; second, to arrest further decay; third, to consolidate the concrete and thereby restore to a large extent the original strength of the structure.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *Supplement, NOVEMBER 1941*

Discussion of a Paper by T. C. Creaghan:

The Grouting of Concrete Structures*

BY R. F. BLANKS, R. B. YOUNG AND AUTHOR

CONVENTION DISCUSSION

R. F. Blanks†—The most remarkable thing, at least to me, in Mr. Creaghan's discussion, was the fact that concrete could be so rotten and still be in service and worth salvaging. Can you give us some idea of just how bad concrete has to be before it can be rehabilitated by this method you have described?

Mr. Creaghan—Well, that depends on the structure. Consider such a structure as the dam at Saint Raphael—it was simply a pile of stone when the hard skin was removed. It seemed like a masonry wall, and a structure like that can be definitely saved; but another structure that has gone—(Mr. Young knows one on the Ottawa River) that seems like a pile of hard mud—we have never found any way of repairing a dam of that type.

Mr. Blanks—You mentioned that you used pressures from zero to several hundred pounds. What is the usual pressure used in grouting structures of this sort?

Mr. Creaghan—About 25 to 30 pounds.

Mr. Blanks—Then I was very curious as to why you used sodium carbonate.

Mr. Creaghan—Well, it is not very popular with engineers, I know, but a small quantity (from the tests we have had) we believe does no harm. Where we used it for temporarily stopping the water, larger quantities can be used. We never leave the strong sodium carbonate

*ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 641.
†Bureau of Reclamation, Denver.

in the structure. We take it out afterwards. In our grouting, we use a small portion—what we consider a safe proportion of sodium carbonate to make our cement set up quickly.

Mr. Blanks—It is an accelerator, then?

Mr. Creaghan—Yes, with enough of it a man can take a handful of cement and sodium carbonate and stop water immediately.

Mr. Blanks—We have used at times a combination of aluminous cement and portland cement to shut off heavy flows of water. That is the purpose of it?

Mr. Creaghan—Yes.

Mr. Blanks—And then you mentioned lubricating agents. I suppose your usual agents for this purpose?

Mr. Creaghan—Yes.

Mr. Blanks—To obtain greater penetration of grout?

Mr. Creaghan—Yes.

Mr. Blanks—In your paper you mentioned that grout travelled as much as 40 feet from the point of entry?

Mr. Creaghan—Yes, in several instances.

Mr. Blanks—You mentioned that in most cases these structures that have disintegrated and which you have been repairing, did not have enough cement?

Mr. Creaghan—Yes, that is true.

Mr. Blanks—Can you be more specific on that? What is the normal cement content of those structures?

Mr. Creaghan—Well, one that we are working on today had four and a quarter bags of cement per yard, but the sand was very fine, and that one we could not grout. Others ran four to four and a half bags, poured very wet.

Mr Creaghan—Some of those old structures are in very good shape. That one at Shawinigan Falls was built in 1901 and will last a great many years yet.

*R. B. Young**—I would like to have Mr. Creaghan enlarge on his "floating caisson." I think it would be interesting.

Mr. Creaghan—This floating caisson is a pet of mine. Some years ago the dam at Gouin Basin, which was at one time the biggest artificial storage basin in the world, had to be repaired from the upstream face. Mr. McCrory, Vice-President of Shawinigan Engineering Co., and I designed this system of unwatering. It consisted of an open-

*Hydro-Electric Power Comm. of Ontario, Toronto.

faced box about 25 ft. long by 22 ft. deep, with a 6-ft. wide working chamber to fit up against the upstream face of the dam, and it worked very well. This caisson could be moved and pumped in about eight hours from the time work ceased on one face until ready to work on another face. On the gate section of the dam, the caisson spanned from outside to outside of the piers, with the bottom extended into the gates. We had to design five different caissons for the straight face and different angles of the dam.

An interesting application of this caisson occurred in the repair of a head gate structure. The upper portion of the piers supporting a platform above the wheels had to be removed and replaced. A hinged floating caisson was placed around the piers and wedged tightly on to the lower solid part of the piers. The weight of the platform was taken on the caisson. Then the disintegrated portion of the piers was cut off and replaced.

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Discrepancies Between the Volume of Fresh Concrete at the Ready-Mix Plant and the Volume in Final Placement*

By Herbert J. Knopel†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The rapid growth of the Ready-Mix Concrete industry has entailed production and service problems. These include technical problems of precision control. The absolute volume method for deducing resulting yield of wet mixes is sound. The wet mass is altered by evaporation losses, by densificatlon due to agitation, and by bleeding through forms or subgrades; these varying with the degree of manipulation. The purpose of this paper is to show where these losses may occur.

INTRODUCTION

The purpose of this paper is to call attention to some practical problems which arise in the commercial production and delivery of ready-mix concrete.

The following three phases involve discrepancies in volume yield:

A. One phase is due solely to lack of care at the job in securing reasonable accuracy in final form dimensions and in the preparation of sub-grades.

B. Another phase involves the variable in volume yield which exists in wet consistencies of concrete, made necessary or demanded by job requirements, when consideration is given to water gain or bleeding due to the natural subsidence of aggregates.

C. A third phase involves inconsistencies in the weights forwarded to the producer from representatives of various Bureaus.

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†Engineer, Philadelphia.

STATUS OF PRODUCER

The Standard specifications for Ready Mixed Concrete of the A. S. T. M. define the measurement of the volume yield of concrete as follows: "The quantity of concrete produced by a given combination of materials may be determined by measurement in a standard measure, or may be calculated from the absolute volumes of the separate ingredients as determined from the weight of each used in a batch and its specific gravity. When the latter method is used, correction shall be made for the free and absorbed moisture content of the aggregates."

The absolute volume method includes all water used in the mixture as part of the yield, and rightfully so, but this may result in disappointments.

The ready-mix concrete plant is called upon to produce and deliver concrete mixtures designed to serve widely different purposes. At times, the plant technician must disregard the existence of improper terminology in qualifying materials and resulting concrete, or any confusion in the improper correlation of the terms "consistency, "slump," "proportions" and "w/c ratio."

The development of Central-Mix Concrete Plants, with advanced methods of production, introduced the sale of the wet mixture by the cubic yard. Unfortunately, due to many contradictory formulas which emanate from various sources, there are those who still believe that concrete should be sold by weight, but the majority believe in the cubic yard as the unit of sale.

A question, therefore, arises. "Shall this cubic yard of freshly mixed concrete include the total water content in the sum of the absolute volumes of the four constituents, or shall the producer take cognizance of the slight shrinkage which may occur due to aggregate subsidence?"

There is general agreement that excess water is needed as a lubricant for placement. This applies particularly to structural mixes to be puddled and not vibrated. In the low slump concrete employed for paving, with 4¼ cu. ft. (32 gallons) or less of water per cubic yard of concrete, the excess is practically negligible.

It is incumbent upon the producer to fabricate and deliver the kind of concrete permissible under the terms of contract. He has a further obligation to govern the quantities of materials used to produce a cubic yard, or multiple thereof, of mixed concrete at the site of the work, but within the limits of specifications and the purchaser's order.

Since the general adoption of proportioning the cement and aggregates by weight, and the water by closely calibrated volumetric measurement, the test for yield has been more consistent. There has been

removed the variable so common to the moist volume bulking of aggregates, because the aggregates are weighed now in terms of dry aggregate and any aggregate moisture is considered as included in the total mixing water.

ERRORS FROM INACCURATE FORMS

In the plan take-off of a job, where the volume of concrete is determined, the number of cubic yards involved becomes a basic integral part of the contractor's bid. If he mixes the concrete at the site of the work, extreme care must be exercised through proper supervision so that the calculated cement factor and the tons of aggregate fall reasonably close to his estimated materials. If there is an excessive overrun by comparison with the plan take-off, he may take his superintendent to task.

Contractors can minimize apparent discrepancies or shortages by exercising care in decreasing the dimensions of their forms, say, ¼ in., or cambering up floor forms ½ in. to allow for deflection due to the dead load pressure of the fresh concrete.

My records show instances where ¼ in. added length of screed blocks for finished floor level, which is approximately the amount of a saw kerf, caused discrepancies of 6¼ per cent on 4-in. concrete slabs; this amounts to an increase in cost of 50¢ per cu. yd.

Other cases show from ¼ to ⅜-in. spread in 12-in. wall forms which caused discrepancies of 2½ to 3 per cent. Speed in striving to effect low labor costs does not offset these discrepancies, because the loss per cubic yard represents ½ man-hour.

On large structural concrete jobs, there are usually two consistencies required—4- and 6-in. slump—according to the mass requirements. Naturally the contractor, by a count of the buggies, feels that he is getting more concrete with the wetter consistency. If the yield is based on 3- or 4-in. slump, a physical check would show a difference of about ½ cu. ft. per cu. yd. in favor of the wetter concrete. He doesn't realize, however, that the wetter concrete ultimately results in less yield because of the excess water's contribution to losses.

On a large reinforced concrete structure of flat slab design, the screeds for finished floor line were set at the column centers. Due to deflection of forms under the dead load, the 8-in. designed slab measured an average of 8½ in. thick, amounting to 6 per cent discrepancy or loss. This is about 50 cents per cu. yd., or 1¼ cent per sq. ft.

On an open cut subway job, where horizontal sheeting was wedged in between the flanges of vertical steel H-beams to act as back forms, errors in alignment caused an increase of 4½ per cent over the volume as computed.

On a large building with structural slab designed as rib type floor construction with tile filler block, the loss as measured by the hardened concrete fins between filler and 2 x 8 in. soffit board amounted to 1½ per cent. This loss, plus aggregate subsidence, made the discrepancy 3 per cent.

My records also show cases where contractors were neglectful in not noting that the specifications called for concrete to be vibrated. This might result in shortages of from 3 to 4 per cent, which, in some instances, would represent the contractor's profit.

The position that the ready-mix concrete producer shall accept place measurement appears untenable. This would involve inspection and quantity surveys by the producer which would be expensive, and frequently would not afford conclusive checks.

In the purchase of ready-mix concrete, the contractor expects close agreement between his plan take-off and the total volume which he receives from the producer as recorded on the delivery slips. On many jobs which have come to my attention, discrepancies and mistakes in the plan take-off have been discovered, but only after the buyer has raised the issue of shortage.

Leaky forms and losses in the job transportation facilities, which are not always known to the contractor, have resulted in ill feeling between producer and purchaser of concrete, because the billing was in excess of the cubic yards summed up in the estimate. Claims for shortage can cause considerable annoyance to a producer and may lead to a bad repute; which, in the final analysis, may not be justified.

EFFECT OF AGGREGATE SUBSIDENCE

With the use of finer ground cements and the variations in characteristics of types of cement, there is introduced an additional variable, namely, the water demand to satisfy a given slump. This can be overcome by supplying concrete with less slump and without a water penalty, due to the higher degree of plasticity resulting from the use of finer ground cements, which is a more desirable adjustment because it facilitates job placement.

The degree to which a plastic concrete mix lends itself to further densification by means of mechanical vibration, may require from 2½ to 4 per cent above the volume yield of fresh concrete after aggregate subsidence, depending upon the cement factor, degree of wetness, aggregates and grading combination.

For example: For a mix of 1-1.65-3.78 by weight and 164.5 lb. absolute volume weight for aggregates, using 5 gal./sk. cement, the absolute volume yield becomes the sum of the following:

3.07	Cu. ft.	cement
6.02	"	sand
13.7	"	coarse aggregate
22.79	"	of solids
4.23	"	water
27.02	"	in batch

Assuming 3 per cent added densification as reasonable expectancy, the water loss becomes 0.81 cu. ft.; and the adjusted volume of solids to produce a cubic yard in subsided condition becomes: 22.79 cu. ft. + .81 cu. ft. = 23.60 cu. ft.

It is misleading to state that "concrete shrinks because of setting." A misconception has resulted from the actions of settlement which occurs in freshly mixed concrete, with the consequent accumulation of water at the surface. However, some of this water at the surface can be re-absorbed by the concrete into voids created as the hardening takes place.

Observations on a number of concrete jobs indicated rather conclusively that in the leaner mixes, such as nominal 2500-lb. concrete, where some water was necessary for added lubrication and mobility, the loss in actual volume yield averaged 1.5 per cent due to aggregate subsidence.

ACCURATE SPECIFIC GRAVITY IMPORTANT

Repeated tests in the laboratory, or in the field, where apparatus is available for securing precise measurements of a batch of freshly mixed concrete, indicate close agreement between the arithmetical sum of the absolute volumes and the physical volumetric measurements of the resulting concrete. To insure this agreement, care must be exercised in obtaining the specific gravities of the aggregates, for the reason that both bulk, and apparent specific gravity determinations will vary as much as 1½ per cent for a given fine aggregate or a coarse aggregate; this also brings into account differences in laboratory determinations.

The weights (per sack of cement) of aggregates forwarded to the producer from representatives of various Bureaus have been noted to vary to the extent of producing an excess or a shortage of as much as 1½ cu. ft. per yd. of fresh concrete.

TESTS OF WATER REQUIREMENTS

To secure data upon which to base a new approach to volume yield, a series of tests was run with a given fine and coarse aggregate. The tests comprised a number of concrete mixes containing from 4 to 5½ sacks of cement per cubic yard with consistencies of 4 and 6-in. slump.

After the proportions, total water and water ratios were found, the batches were re-run and the concrete placed and puddled in a closely calibrated measuring unit 12-in. wide, 12-in. deep and 30-in. long, to check yield and measure water gain or bleeding. The surface water was measured at intervals of 15 and 30 minutes, and calculated in terms of volume and percentage of the mass.

As different types of aggregates will vary in proportions, it appears unnecessary to mention all of the data secured; the following enumeration should serve to illustrate basic results:

Coarse Aggregate (Top size, square opening)	*Water per cu. yd.* (after aggregate subsidence)
2 in.	4.4 cu. ft. (33 gal.) to 4.6 cu. ft. (34.5 gal.)
1 in.	4.5 " (33¾ ") to 4.7 " (35¼ ")
½ in.	5.0 " (37½ ") to 5.2 " (39 ")

It will be noted that excess water tendency decreases with the decreased diameters of the particles.

The results tend to show that for a 2-in. top size gravel, regardless of a cubic yard batch requirement of 4.8 cu. ft. (or 36.0 gal.) of water to produce a desired consistency or slump, only 4.4 cu. ft. (or 33.0 gal.) of water would remain in the concrete mass after aggregate subsidence.

The difference of 0.4 cu. ft. (or 3.0 gal.) of water would be the amount lost in bleeding during aggregate subsidence. As previously mentioned, the various coarse aggregate sizes would result in varied amounts of total water after aggregate subsidence.

POLICIES REGARDING OVER-SANDING AND EXCESS WATER

It is found that sanding up mixes above the optimum ratios of fine to coarse aggregates necessary for normal requirements (as an aid to placement in narrow, heavily reinforced sections, in pump transportation, or where tremies are employed) will lessen bleeding and slightly increase volume yield.

If the better method is used, of altering the mix by a reduction of the coarse aggregate, the yield is lessened; this results in a discrepancy, or shortage with an increased cost to the contractor.

Some specifications incorporate a clause prescribing the latter method, but no provision is made for added compensation, which confuses the ready-mix operator in billing his customer.

To avoid what will appear as a discrepancy, or shortage, in yield, another policy adopted is to notify the contractor in advance as to the increase in cement content in adjusting the new proportions and the weights to guarantee a cubic yard.

The question has been raised as to whether, due to agitation or temperature, there is any apparent volume shrinkage in ready-mix con-

crete. Tests have shown that increase in density is in ratio with the amount of water taken up by the cement, and the apparent shrinkage is the measure of the free water so absorbed.

The total water used at the plant to insure a desired consistency may be reduced as much as 1½ gal. per cubic yard by absorption into cement paste en route to the job. What seems to occur is that, as large particles of cement are broken down, by grinding action, some water is taken up in producing a more effective cement paste. Experiments made by slowly agitating definite amounts of cement and water for periods up to ½ hour, seem to justify this statement.

A further series of tests showed that samples of concrete taken at the mixer after 1½ minutes mixing time were 1 to 2 pounds lighter than samples taken from the same batch upon arrival at the job. This increased density will amount to between 0.5 per cent and 0.8 per cent.

Based on the data thus secured, a policy was adopted which fixed the maximum volume of water per cubic yard to be included in computing volume. Any water in excess of this amount is not considered in the volume yield. The mix design is adjusted by compensating an equal absolute volume of solids for the excess water above the standard.

POLICY REGARDING VIBRATED CONCRETE

Two schools of thought prevail as to the method of providing additional solids to bring vibrated concrete up to any unit of volume. They are:

a) The amount of cement per unit of volume is to be kept constant regardless of whether the concrete is placed by puddling or by mechanical vibration. This means that the additional solids per unit of volume, to allow for subsidence caused by vibration, would consist of the proper proportions of fine and coarse aggregates.

b) The proportions of cement, fine and coarse aggregates are to remain constant regardless of the concrete being placed by puddling or vibration. This means that the additional solids per unit of volume, to take care of subsidence caused by vibration, would consist of holding the proportions of cement, fine and coarse aggregates constant.

The producer cannot make further allowance for greater densification of concrete induced by mechanical vibration; this feature is properly under job control. Special arrangements should be made between the purchaser and the producer when mechanical vibration is employed for compaction.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the* JOURNAL *Supplement concluding this Proceedings volume

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Discussion of a Paper by Herbert J. Knopel:

Discrepancies Between the Volume of Fresh Concrete at the Ready-Mix Plant and the Volume in Final Placement*

BY HERBERT J. WHITTEN, A. A. LEVISON, JOHN C. SPRAGUE AND G. W. HUTCHINSON

CONVENTION DISCUSSION

Herbert J. Whitten†—I would like to make one or two observations of our own day to day experience in fabricating and delivering concrete. They will be more in the line of emphasizing the points which should be considered in avoiding trouble. On the matter of yield: I have not found that there was anything mysterious about the yield of concrete. Most of our trouble is due to the conditions at the site of the work in the forming operations and in the grading operations. These operations are not absolutely accurate, nor are they always in accord with the plans for the job; furthermore, I think that no one expects absolute accuracy. In our operations, we maintain an organization which includes field representatives who visit the work which we are serving and familiarize themselves with what is going on. When discrepancies in yield develop, we usually have ample information with which to determine where the trouble lies, in order to reach a satisfactory conclusion. Another one of our difficulties, as stated by Mr. Knopel, is that frequently some architects, engineers or public authorities having supervision over a job, give us arbitrary quantities to use per cubic yard. We find that these figures have not been carefully developed to produce a cubic yard. Our policy, invariably, is to insist upon the necessary adjustment of such figures before we begin to serve the job. With the increasing use of vibration in placing con-

A.C.I. JOURNAL, June 1941; *Proceedings*, V. 37, p. 649.
†Warner Co., Philadelphia, who in the author's absence, presented Mr.Knopel's paper.

crete, numerous cases have come to our attention where the cubic yard of fresh concrete furnished by the ready-mix operator results in a lesser yield. The only difficulty which presents itself here is to have a mutual understanding with the user, before the work goes ahead, that the volume of concrete being supplied is based upon the yield of fresh concrete as produced at the mixer. There is another point which frequently has come to my attention—a miscalculation of needs on the smaller jobs, due to lack of acquaintance with the use of ready-mix concrete, quite frequently results in a surplus, and at the end of the pour considerable concrete is sent back frequently to the plant. I would like to emphasize Mr. Knopel's reference to the use of the total amount of water in calculating the absolute volume of a cubic yard. We have learned by tests and experience that a variable amount of water does not contribute to yield. Mr. Knopel has given an example in figures, and I would say that a good average is four and a half, or 4.6 cu. ft. of water.

*A. A. Levison**—Realizing that Mr. Whitten is a practical operating man, I would like to ask him, whether, in the controversies that result between the shippers and purchasers of concrete, checking the individual weights of the materials going into the various batches would be of any advantage?

I ask that because of an experience that we encountered in Barberton, Ohio, where there was serious trouble when carload shipments of soda ash were reported back by the purchaser as short in weight—shipments weighed over railroad scales?

The solution adopted was a batch type weigher which automatically weighs and records the printed weight of each batch on a paper tape. The batches were measured out in 2000-lb. lots, 30 batches to a car, and a permanent record of the 30 batches was printed on a tape. One copy of this printed record went with the invoice for the carload of soda ash. This type of equipment is expensive, although when properly developed it gives good service.

In concrete, on some of the larger projects, the engineers require automatic graphic records of the measurement of aggregates, cement, and water in connection with the making of concrete. The question came to me as Mr. Whitten was discussing this problem, as to whether the addition of this type of equipment to a central mixing plant or a truckmixing plant would help in any way toward the solution of some of the problems connected with the ready-mixed concrete industry.

*Blaw-Knox Co., Pittsburgh

Mr. Whitten—Of course any mechanical or automatic record of procedure in concrete plants is good. For many years now we have pursued a policy of keeping a definite record of every batch, and there have been occasions when that record has assisted materially in resolving the controversies on yield. However I feel that the main point in this connection is to be very careful and very certain in the beginning on every job served, that the weights of the formula supplied or in use, specified or proposed by the ready mixed operator, unquestionably and without doubt do produce a cubic yard, and then to maintain sufficient contact with the user so that any questions arising might be amicably settled by a review of the facts on both sides.

BY JOHN C. SPRAGUE*

On page 654 Mr. Knopel says: "It will be noted that excess water tendency decreases with the decreased diameter of the particles." That this condition obtains is not quite clear to the writer. Based on the 36-gal. initial water content for the 2-in. aggregate, the slump is approximately 6 in. According to this, the 1-inch material must have required about 39 gal. of mixing water per yard of concrete; and the ½-in. aggregate, about 43-44 gal. to produce this same slump. Hence, while 3 gal. of water were lost in the case of the 2-in. aggregate concrete, apparently nearly 5 gal were lost for the 1-in. and about 6 gal. lost for ½-in. aggregate concrete. Thus the percentage of bleeding for the three sizes would be of the order of 8½ per cent, 13 per cent and 14 per cent respectively.

The tendency for concrete to bleed varies greatly of course, depending upon the percentage of sub-sieve fines in the mixture. A few years ago the writer conducted accelerated water-gain or bleeding tests in which various types of sub-sieve fines were incorporated in the concrete; the results of which were reported as: "Evaluating fines in concrete on a bleeding test basis."† For 2 in. aggregate concrete having a slump of 3-4 in. the per cent bleeding varied from ½ for a very finely pulverized admixture to 10 for concrete containing virtually no sub-sieve fines at all. For concrete containing 2 per cent sand passing the 100-mesh sieve, the per cent bleeding was 7. This checks approximately with the water loss reported by Mr. Knopel.

By and large, the suggestions offered by Mr. Knopel seem to be very practicable and to the point. Certainly some excess yield is necessary in the plastic mixture if a yield of 100 per cent is to be obtained in the hardened concrete. The writer has advocated a flat

*Development Engineer, Dravo Corp., Pittsburgh.
†ACI JOURNAL, Sept. 1936; *Proceedings* V. 33, 1937, p. 29.

excess yield of from 1 to 1½ per cent to cover losses due to subsidence of solid particles, air voids and other contributing factors.

BY G. W. HUTCHINSON*

Mr. Knopel has presented a subject of increasing importance. The difficulties encountered in establishing a fair and satisfactory basis of payment for ready mixed concrete have been recognized ever since the industry originated.

The problem might not be so difficult if it were possible to secure the cooperation of the purchasers and establish a universal standard of unit measurement. The range in consistency for concretes, placed by vibration as one extreme and by hand tamping as the other, causes the water content to become more important than implied by the paper.

In the light of little experience, standards many times mean nothing. The policy that the "customer is always right" will lead to unnecessary expense and tend to depreciate the quality of concrete. The problem is obviously one in which the producer must take the initiative. By doing this, he not only helps himself, but can be of invaluable service to the purchaser.

Although the author refrains from stressing the procedure when vibration is used to place the concrete, it should be considered. Concrete suitable for placing by vibration must contain less water than that required for hand placing if all advantages of vibration are to be secured. The ready mixed concrete producer must be in a position to furnish concrete for placing by either method and for all project conditions. A 6-in. slump concrete might be too dry in one case and one of 3-in. be too wet in another. As regards the producers' direct interest, the matter of how the concrete is to be placed is of no concern —his interest is only in the amount of water necessary to produce a placeable concrete under any given conditions. The range in water content between the wet and dry consistencies might amount to as much as 5 to 6 per cent of the volume of concrete.

Although several methods have been proposed as a basis for payment of ready mixed concrete, variables occur which cause controversy. A method which eliminates the most and more important of these would appear to be the best. A method which regulates the sale price, based upon the cost and amount of each of the solid ingredients, computed by individual absolute volumes, would appear to be rational. The unit of measurement would be the absolute volume of the solid materials in the batch rather than the cubic yard, etc. The limits of

*Raleigh, N. C.

error would depend upon accuracy in determining the specific gravities of the solid materials and the limits of practical plant and aggregate control.

The solid ingredients of ready mixed concrete represent the only constituents of appreciable cost. The cost of water is relatively negligible. It can be absorbed in the proportioning, mixing, placing, and delivery costs of the concrete. It would be eliminated in its influence on the direct sale price of the concrete whether present as absorbed, adsorbed, or free water. Concrete made from identical amounts of given solid materials would sell for essentially the same price whether furnished with an 8-in. or a 2-in. slump. It would then make no difference, as regards basis of payment, whether or how much water was contained in the concrete in place or went through the forms, etc. Definite amounts of solid ingredients could probably be permitted to constitute each batch in practice, regardless of the water content.

By this method the producer can just as easily establish data to meet any strength, mix, or other specification requirement. He can permit the customer to make his own selection from a variety of mixtures and consistencies. For a given strength, he can select a leaner mix of drier or a richer mix of wetter consistency. Regardless of the one selected, the basis for payment will be consistent with the cost of the solid ingredients. Such procedure would not preclude the use of trial mixtures until inspection plus field or laboratory test data dictated the one which produced best results.

A typical example of a cost chart is given in Fig. A. Arbitrary costs are taken for the solid materials. No allowance for variation in the fine-coarse aggregate ratio for different cement contents, or for other features of mix design, etc., is made. It merely illustrates a means for comparing the unit costs of various concrete mixtures based upon the absolute volumes of the cement and aggregates. To this would be added the cost of proportioning, mixing, delivery, overhead, profit, etc. Such costs could be either added as a lump sum, regardless of mix, or be on a percentage basis. The water content, governing consistency, is left entirely to the purchaser.

It would make no difference whether the water was added at the plant, during transit, at the job, or by any combination. With practical control of consistency, the variation in the water content would be inappreciable. The slump test could be used as a specification. The judgment of an experienced operator would probably be as accurate as this test in making practical adjustment during normal operation.

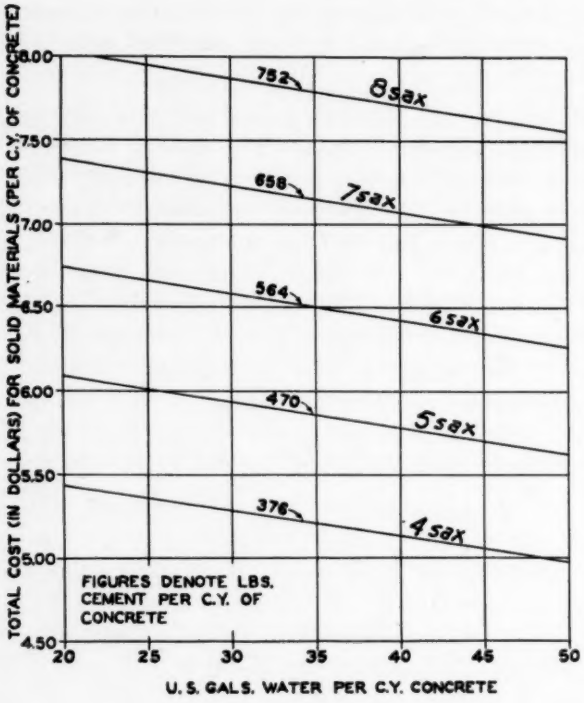

FIG. A

The suggestion of this, or similar method, is prompted by the fact that, when the wide range in water content between the wetter of the hand placed and the drier of the vibratory placed mixture is realized, the variation in, and difficulty of computing, actual cost becomes an important practical item.

The fact that the manufacturing process of ready mixed concrete lends itself to technical design of mix and accurate control, leads one to wonder if it is for convenience or to satisfy the problem purchaser that the product is still marketed as described, *i.e.*, under antiquated proportions such as 1:2:4.

AUTHOR'S CLOSURE

I want to clarify my remarks in the first paragraph, of which Mr. Sprague asks for an explanation; so, I will amplify that statement. It is predicated on producing comparable concrete in the ½-in. size aggregate with the 1-in. or 2-in. aggregate. With this base and the same water cement ratio, I have found that the bleeding is approximately the same in terms of gallons or cubic feet and since more

water is required in the smaller size aggregate, the percentage decrease is less with the finer aggregate than with the larger aggregate.

I want to add that I recognize only too well what Mr. Srpague has pointed out: the value of the fines in concrete sand, provided that it is consistent with the particle shape and grading. I usually recommend a flat excess of 2 per cent, and in general use 22.5 as absolute volume for the solids, although there is justification for using 22 cu. ft. of solids where very small aggregate is employed. We must recognize, of course, that even 3-in. and 4-in. slump concrete will bleed, so that 27.54 cu. ft. or 2 per cent excess, should be definitely associated with a well-designed mix of workable concrete.

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Placing and Finishing Pavement Concrete*

By H. F. Clemmer†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Discusses the possibilities of increased speed and performance in concrete highway construction through the use of larger equipment but emphasizes the coordination of machinery for increased output in all the operations. The increased output of a paver can be utilized only with comparable increases in performance with placing and finishing equipment.

With the advent of the enormous national defense program inaugurated by Congress, and with "speed" the by-word, it is realized that there will be a definite demand on the highway federation to transform our peace time highway system to one which will meet the defense requirements adequately.

Conferences between army engineers and federal highway administrative officials will determine the program needed, but it will be the responsibility of highway design and construction engineers, in cooperation with the highway contractors, equipment manufacturers, and materials producers to speed the work to its important completion.

The construction of the super highway from Harrisburg to Pittsburgh and of less extensive, but equally vital thoroughfares in other sections of this country, have been a stimulus to the engineer toward possible accomplishments which may be attained under large scale construction programs. The potentialities have interested the manufacturer of highway construction equipment and have emphasized the economical use of larger equipment of all types—excavators, hauling and grading equipment, mixers, spreaders, vibrators, etc.;

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†Engineer of Materials, District of Columbia.

all of which tend to extend the daily output or accomplishment to new limits.

The constant demand by the engineers for economy in construction and improvement in quality, through better control and better methods of operation during the last twenty years of road building, has resulted in the progressive development of more economical highway construction equipment. In view of the striking developments in mixing, placing, and finishing equipment during the past few years, and the fact that the correlated efficiency of these procedures really determines the daily output of the paving crew, the following discussion will be confined mainly to developments in paving equipment. In considering such developments, the impracticability of greatly increasing the concrete output without increasing placing and finishing capacities simultaneously, is apparent. From a practical standpoint, therefore, equipment referred to for any specific purpose should be utilized in conjunction with equipment developed to an equivalent degree for any subsequent operation. For example, the capacity of a 34-E dual drum paver could not be utilized efficiently if operated with finishing and placing equipment which is barely adequate when used with a 27-E paver.

PAVERS

Subsequent paving operations must of necessity be geared to the capacity of the mixer, the maximum output of which can be readily estimated. In view of the normal job delays incurred, it is evident that the efficiency of any mixer is considerably less than the theoretical maximum.

The following table, however, serves to illustrate the relative maximum capacities of respective types of mixers now in use, for mixing periods varying from one to two minutes. These calculations have been based on a total charging and discharging time of 15 seconds per batch for the single drum mixers and of 10 seconds per batch for the dual drum type.

Mixing Time	27-E	34-E	34-E Dual
1	48	60	112
1¼	40	50	95
1½	34.3	42.6	81
1¾	30	37.5	72
2	26.6	33.5	64

From this table it is evident that for the same mixing time the 34-E single drum paver has a capacity 25 per cent greater than the 27-E, while the 34-E dual drum mixer (in which the concrete is mixed for one-half the specified mixing time in one drum and then transferred

to a second drum for completion of the mixing) has a capacity approximately 130 per cent greater than the 27-E, and 85 per cent greater than the single drum 34-E. The manner in which this increased capacity has been utilized is evident from the comment regarding the Pennsylvania Turnpike project, to the effect, "The typical paving contract used a 34-E dual drum paver." General paving schedules approximated 300 ft. of 12 ft. pavement, 8 in. thick, per hour. In comparison, the 27-E paver might have been expected to permit a paving schedule of 130 ft. per hour.

Mixing efficiency of the respective types of mixers is a problem demanding study on the part of the engineer and the equipment manufacturer. For example, some states have specified increased mixing periods for mixes of the greater capacity; typically, mixing for 1¼ minutes has been specified for a 27-E paver while 1½ minutes has been specified for the 34-E. The effect of such increase in mixing time is, of course, to nullify the greater output of the larger mixer. This problem is of even greater significance when it is realized that mixing times commonly specified vary from one to two minutes, which range alone is sufficient either to double or halve the output of a mixer.

SPREADING EQUIPMENT

As previously discussed, efficient utilization of improvements in one operation necessitates corresponding use of improvements in other paving practices. Spreaders are being developed primarily to cope with the increased mixer capacity and to obtain a more uniform distribution of the concrete. They do, however, serve to particular advantage in the placing of harsh mixes suitable for use with vibratory equipment, the nature of which makes efficient handling by workmen impossible. Spreaders are used to strike off the concrete prior to placing the steel reinforcing and again at the surface prior to the final finishing.

Two types of mechanical spreaders have been developed within the last year and a half and were extensively used on the Pennsylvania Turnpike projects. In regard to the economical features of spreaders it is of interest to note the comments of turnpike engineers; "the action of the spreader enables the engineer to design a dense mix with a minimum amount of water which cannot be economically spread by hand . . . the spreader has become a valuable piece of equipment in the opinion of both the engineer and the contractor. It has demonstrated that the quality of concrete can be improved and at the same time cost of production to the contractor may be reduced."

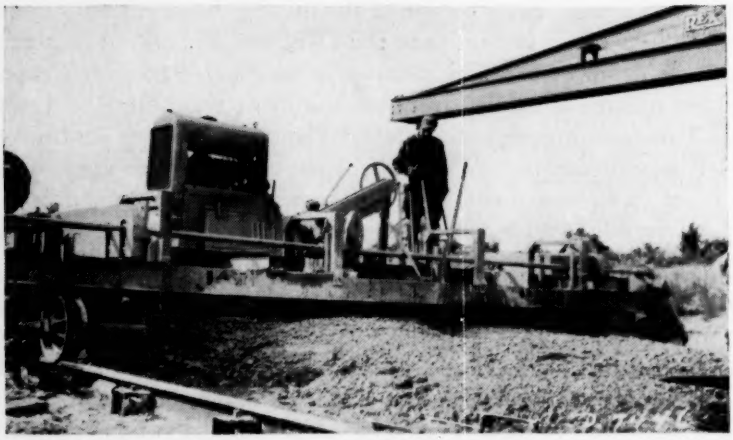

FIG. 1—MECHANICAL SPREADER OF THE HORIZONTAL SCREW TYPE

In addition to the placing efficiency effected by mechanical spreaders, they serve greatly to improve the uniformity of the concrete and to minimize the variations between batches, which, in effect, results from "remixing" of the concrete after deposition on the sub-grade. In view of the inclination on the part of workmen to distribute the more workable portions of deposited mixtures, and not the compacted portions, non-uniformity between batches may be expected to be increased by hand methods; similar conditions likewise exist in those cases where coarse aggregates may have segregated. Job experience with mechanical spreaders has indicated the utility of such equipment in minimizing the effect of such factors which contribute to the non-uniformity of concrete batches.

It seems probable that the lag between the proven value of vibration and its universal adoption as standard practice has, in effect, resulted from the economical considerations involved in hand placing of the harsh, low-slump mixes, characteristic of concrete suitable for use with vibratory equipment. These developments in placing equipment may be expected to further the adoption of vibration by road building organizations.

VIBRATORY EQUIPMENT

The value of vibration in imparting greater durability and uniformity to concrete has been demonstrated on many projects, in fact the Public Roads Administration has followed the application and progress of vibratory methods in highway construction for approximately ten years. The economy and quality properties of concrete

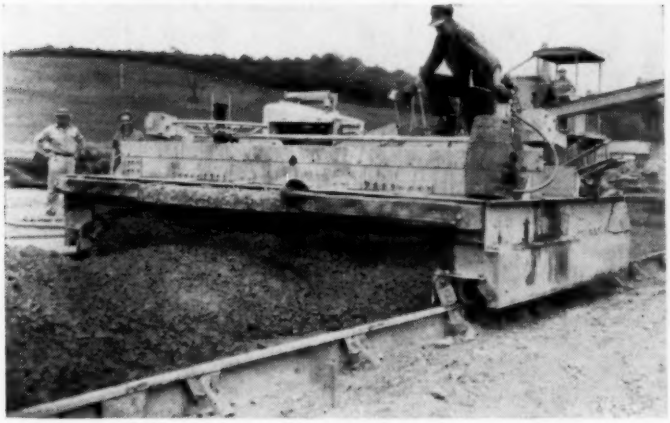

FIG. 2—MECHANICAL SPREADER OF THE BLADE TYPE

as effected by vibrators of the bull nose screed and pan types, may be noted in a study by the Public Roads Administration reported in the April, 1937 *Public Roads.*

1. For a given water-cement ratio a saving of approximately 10 per cent in the amount of cement can be effected by the use of vibration without sacrificing strength and uniformity.

2. For a given cement content an increase in flexural and compressive strength of approximately 10 per cent can be obtained by the use of vibration.

Vibrators of many types are being utilized throughout the country with at least six major companies manufacturing equipment differing in many physical respects, although designed to effect the same general improvement in quality of concrete attendant with the efficient placing and compaction of lower slump mixes. The various types of equipment presented may however, be divided into two general classes, viz. the internal and surface types. Discussion as to power, frequency, amplitude, time of vibration and distance affected, as produced by the different types of equipment offered, should, of course, be given consideration in each project. It is believed, however, that the manufacturers have had ample experience with the various types of mixtures to be encountered over the country, and that they have accordingly recognized the necessity of designing equipment for efficiently placing concrete of the maximum degree of harshness.

The Highway Department of the District of Columbia in cooperation with the Public Roads Administration has just completed a series of tests with two types of machines for vibration of pavement concrete. Both types were of the internal vibratory tube design,

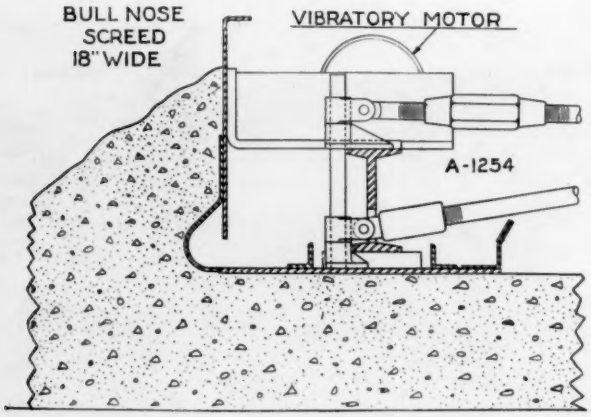

FIG. 3 AND 4—BULL-NOSE VIBRATORY EQUIPMENT IN CROSS-SECTION AND IN OPERATION

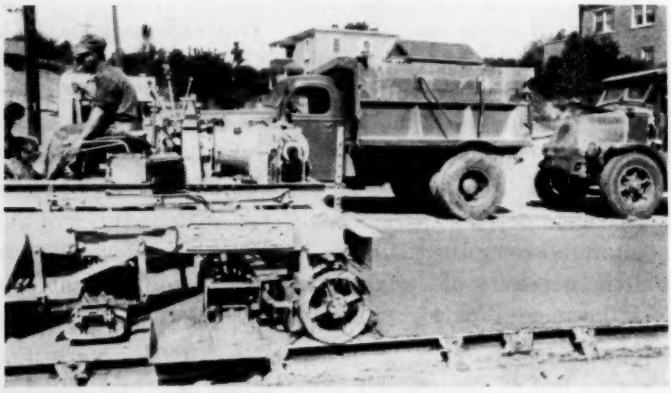

FIG. 5—PAN TYPE VIBRATOR SHOWING VIBRATORS SUSPENDED JUST AHEAD OF THE REAR SCREED

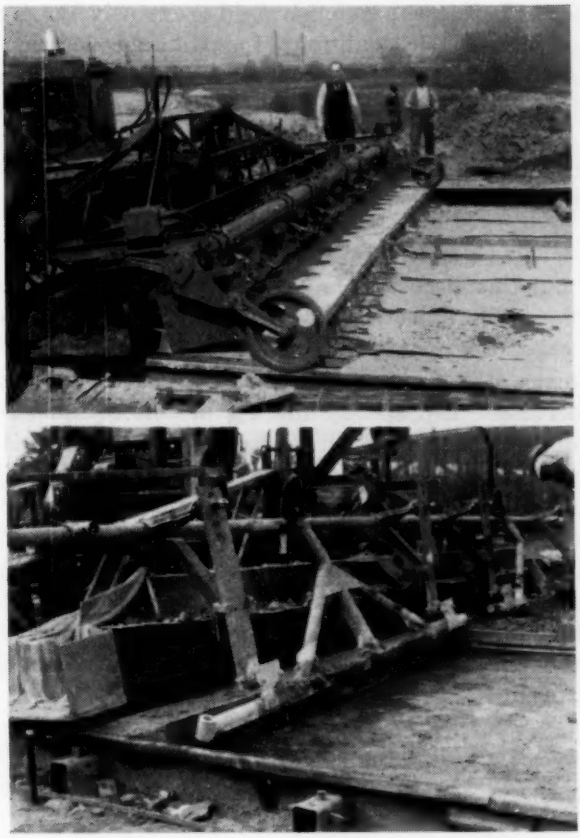

FIG.. 6 AND 7—INTERNAL TUBE TYPE VIBRATORS STUDIED ON THE JOINT PUBLIC ROADS ADMINISTRATION—DISTRICT OF COLUMBIA EXPERIMENTAL PROJECT

attached to, and extended in front of, a standard finishing machine. The results have not as yet been fully analyzed but it is evident they check the general conclusion as to the value of vibration reported by other investigators in *Public Roads* and in the proceedings of the Highway Research Board.

In addition to providing for the placing and compaction of harsh mixes, vibratory placement of concrete, as in the case of spreading equipment, has been effective in improving uniformity.

FINISHERS

The latest development in finishing equipment has been confined principally to the longitudinal mechanical finishers to take the place

of hand floating processes previously used so extensively. The economy of such equipment under conditions of maximum concreting operations (300 feet of 12 ft. pavement per hour) is readily apparent since placing such a volume would necessitate several crews solely engaged in this activity. It is possible with the light longitudinal float and one operator to maintain pace with the mixing, spreading and placing equipment capable of operating at the rate noted above.

CONCLUSIONS

Throughout this discussion an endeavor has been made to illustrate the extent to which respective types of equipment have been developed and their capacities materially increased. From a practical standpoint, however, it is the problem of the contractor and the engineer alike to coordinate and specify the use of this equipment to obtain the maximum overall efficiency. Significantly therefore, it should be emphasized that it is impracticable to utilize increased capacity efficiently in any operation without developing other paving operations to a comparable degree. The potentialities of the coordinated use of modern equipment are astounding, when viewed in terms of maximum capacity prevailing several years ago, and although we have reached a high plane in economy and quality in highways construction, a little thought in retrospect will make us realize the possibilities for future improvements.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL Supplement concluding this Proceedings volume

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Discussion of a Paper by H. F. Clemmer:

Placing and Finishing Pavement Concrete*

BY A. A. LEVISON AND LION GARDINER

CONVENTION DISCUSSION

A. A. Levison†—The last paragraph of the paper especially attracted my attention. Aside from the views of the various developments of pavement equipment shown in the slides, which have to do with the application of the equipment to the job, the tendency in highway concrete is to utilize dryer mixes, for reasons with which we are acquainted. These dryer mixes are harsher and more difficult to handle than concrete mixes used for the same purpose in other years. There have been examples of projects where the dryer, harsher mixes have been used and great difficulty has been experienced in consolidating and obtaining a satisfactory surface finish on the concrete slab, due principally to the fact that the proper type of equipment was not used to handle the kind of concrete involved.

For example, on the Pennsylvania Turnpike we found on a number of the projects a slump as low as 1½-in. or even less, and that the concrete was rather difficult to handle unless proper and adequate equipment was provided for that purpose. It was fortunate on the Turnpike that in practically all cases the contractors did provide the proper equipment. The chief item of equipment involved in the Turnpike construction for the dry, harsh, crushed stone concrete that had not been widely used prior to the Turnpike project, was the concrete paving spreader. That type of concrete does not lend itself readily to manual handling and spreading of the subgrade. It is very difficult for the laborers to penetrate the pile of concrete as it is discharged from the bucket of the paving mixer with a hand shovel, and as a result the laborer does more or less of a scraping job, scraping

*ACI JOURNAL, June 1941; *Proceedings* V. 37, p. 657.
†Blaw-Knox Co., Pittsburgh.

the stone off the top of the pile and pushing the stones around or shoving them ahead as the case may be.

The mechanical spreader performs that function in an efficient and desirable manner, replacing to a large extent the hand labor that was formerly used for the spreading of concrete on the subgrade.

No vibration was used on the Turnpike paving concrete; it might have been used, but it was not and they were able to obtain through the use of the spreader and sufficient finishing machine equipment behind the spreader a satisfactory consolidation of concrete and a satisfactory finish. The spreader assisted the finishing operation by spreading a uniform bed of concrete for the finishing machine operation.

Some of the state highway departments specify compaction of the concrete by means of vibration. It is logical to assume, I believe, that the use of vibration will become more widespread through the various state highway departments' specifications as time goes on. We can judge that from the popularity of the use of vibration in mass concrete and in structures. It has become customary to use vibration for compacting concrete, and I think we will all see the day shortly when vibration will be more or less widely specified by state highway departments for the construction of concrete paving slabs.

Here again the selection of the type of equipment is extremely important and vital, for the handling of the type of mixes that are designed and used for vibrated concrete pavement. Manufacturers are doing what I believe is a splendid job in developing types of equipment for handling and vibrating paving concrete.

The pictures shown in the slides of Mr. Clemmer's paper illustrate many of these types of equipment. The picture shown of the combination of the spreading and vibrating equipment, which permits both of these operations to take place in one passage of the machine over the concrete, also seems a logical development of equipment for concrete paving operations where the vibratory method is used. I think that a paper of the type presented by Mr. Clemmer is most interesting to the membership of the American Concrete Institute, because it illustrates the ways and means of accomplishing the job, in this case highway paving, after an engineer has specified the type of mix that is to be used, and to arrive at a certain quality and density of the concrete and perfection of surface finish.

I hope that the future programs of the Institute will include more papers of the type that will show how the field operations progress and what developments of equipment are available for the desired results.

*Lion Gardiner**—Having heard so much recently as to the splendid progress records on the Pennsylvania Turnpike and other important road jobs, it may be of interest, momentarily, drop back to 1918, the year that Illinois started its large concrete road program.

At that time if a contractor placed and finished 350 to 400 ft. of 6-8-6, 18-ft. road in a day's work, he was well satisfied. Making a comparison today, based on the use of a single drum 27E paver, an Illinois contractor with a good organization would expect to complete 350 to 400 ft. of 20-ft. 9-7-9 slab in a little over three hours, utilizing fewer men in his actual mixing and finishing operations.

It is further interesting to note that his 1918 investment in paver and finishing machine would be sufficient to purchase an up-to-date 27E paver and finisher, to produce three times the amount of pavement in the same elapsed time. This 1941 equipment would outlast that of 1918, 3 or 4 to 1, and be more reliable from the standpoint of continuity of service.

This progress in equipment design, of course, applies to other items in his complete plant but this direct comparison does indicate what part the manufacturer and his design department has played in the great progress of the last 20 years.

One other comparison may be of interest. Based upon the square yard bid price of early 1918, the Pennsylvania Turnpike Commission would have paid almost double the price per square yard of concrete, plus the cost of reinforcing steel, joints, finishing refinements, etc., not used at that time.

The new developments referred to by Mr. Clemmer have all played an important part. The 34E dual drum paver practically doubles the 27E single drum output and thus it may be readily recognized that balanced finishing and spreading equipment were required.

The remixing screw type spreader had been in use since 1932 in connection with truck mixer operations delivering as high as 100 cu. yd. per hour in a 10-ft. lane. The unit was not used with paving mixers until the advent of the dual drum pavers and the great increase in lineal feet per hour placed, particularly in half-width construction.

It may be stated generally from an operating standpoint that a mechanical spreader shows about an even break economically when working in conjunction with a No. 27E single drum paver, on an eight inch 10-ft. slab. This is not so true on a 20-ft. slab unless there is the harsher vibratory concrete to deal with. However, when a dual drum paver comes into the picture, its economy is evident on any width up to 25 ft. for which such spreaders are available, but irrespective of the

*Vice-President The Jaeger Machine Co., Columbus, Ohio.

operating economy, its ability to handle harsher mixes, to leave a more uniform surface texture, definitely increases the capacity of the finishing machines. With a spreader and one transverse finisher in combination, 250 to 275 ft. of narrow lane and 200 to 225 ft. of 22-ft. slab per hour can be handled satisfactorily in the usual consistency of 2 to 3-in. slump.

These capacity figures have general reference to the 1940 or later model finishing machines which have been redesigned to operate at about 50 to 60 per cent increased speed.

I wish to interject a further reference to a new paver size, namely, the single drum 34E, having 25 per cent increased capacity over the 27E. In some states this new unit has no advantage for the contractor, as due to the extra quarter yard of material, extra mixing time has been imposed. It is thought that this increase has come automatically because in most concrete specifications, added mixing time is required in a 2-yard mixer over a 1-yard mixer. It is suggested that engineers study to correct this penalty which in this situation does not seem warranted by several tests and investigations already made.

Mr. Clemmer has made reference to the application of vibration on road slab and perhaps a little history would not be amiss at this point. It was in 1929, I believe, that F. R. McMillan, of the Portland Cement Association suggested to me at an American Concrete Institute meeting that we attempt such an experiment. In 1930 both in Ohio and Texas, the first machines with vibratory screeds were tried out. The initial results led the Bureau of Public Roads to make its extensive tests of 1931 which were repeated in 1934. In 1932 Missouri, Illinois and Michigan all laid one or two miles of this type pavement, Illinois repeating in 1936 and Ohio and one or two other states making similar investigations. To the best of my knowledge, the conclusions as to economy and quality were very uniform in the case of every investigation. The question which I now bring up and so often ask, is—why, with these data available for nearly ten years, have but eight or nine states made use of it. Illinois, I understand, is now writing vibration into its specifications, making a total of about ten highway departments which recognize the advantages. One answer may be that the mechanical spreader will remove the problem of spreading concrete suitable for vibration.

The longitudinal float has contributed its share to the higher speed finishing demanded in the present day operations and all of this equipment is today at work hurrying to readiness the large airport program already under way.

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Investigation of Causes of Delayed Expansion of Concrete in Buck Hydroelectric Plant*

By H .A. Kammer†
and R. W. Carlson‡
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The concrete in Buck Hydroelectric Plant, built in 1912, gave first noticeable evidence of expansion in 1922. The expansion continued until at the date of this writing the dimensions have increased by about a half per cent, and potential growth remains.

No serious expansion was observed in another hydroelectric plant built only three miles away at the same time with the same cement, but with a different coarse aggregate. The Buck coarse aggregate was crushed phyllite (near slate) and the other was crushed gneiss.

Comparable expansion was observed in certain accelerated laboratory tests on concretes made with the Buck aggregate but with modern cements. The laboratory specimens expanded only where a high-alkali cement was used or where alkali was added to a cement. Also, up to the age of one year, the specimens expanded only when stored at an accelerating temperature (110° F. in these tests).

It is concluded that the expansion is due to a reaction between the aggregate and the alkalis in the cement, probably to form sodium or potassium silicate, and that test specimens stored at 70° F. will expand in due time.

The Buck hydroelectric plant on the New River in Carroll Co., Virginia (Fig. 1) is owned by the Appalachian Electric Power Co. It was constructed in 1912. The dam and the substructure of the power house were made of concrete, which according to photographic records, was poured wet according to common practice at that time.

The coarse aggregate was manufactured at the site and crushed from a native rock known as phyllite, a material which is intermediate

*Received by the Institute April 16, 1941.
†Construction Engineer, American Gas & Electric Service Corp., New York City.
‡Associate Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge.

FIG. 1—BUCK HYDROELECTRIC PLANT AS IT APPEARED LOOKING UPSTREAM APRIL 1, 1941

between slate and schist. The Buck phyllite is apparently sound under all ordinary conditions. It has a low thermal expansion of less than 5 millionths per degree F. It has negligible change in volume due to wetting and drying. A minor fault is that it breaks into flaky particles and thus causes high water requirement in concrete. The chemical analysis is included in Table 1. The fine aggregate was river sand believed to have been satisfactory in all respects.

The cement was obtained from a mill in Fordwick, Va., and had no unusual composition according to routine analyses. The mill is still operating but now uses a different source of clay. Therefore, the alkali content of the cement, which was not included in the early records, cannot be estimated reliably from later analysis. There is some evidence in addition to that presented below, to indicate that the alkali content was high.

About 1922 the operating personnel at the Buck power plant began to notice a small crack developing around the periphery of the pit liner of one of the generating units but they did not consider it significant, assuming that some cracking was to be expected in any concrete structure. Subsequent observations showed more cracks, which seemed to increase in size and extent, and during a routine inspection of the scroll case of unit No. 1 it was observed that some of the stay vanes of the cast iron speed ring were broken.

About this same time some of the turbine runners began to bind on the sides of the speed ring and the runners were pulled from the pit in an attempt to disclose the trouble. When the machine was reassembled it was found that the seat for a thrust collar had to be cut deeper in order to line the runner properly with the speed ring. This suggested a general rising, or expansion, of the concrete.

During periodic observations, a crack was found to be developing around the periphery of each of the three turbine pits and these cracks were gradually growing larger. From time to time, it became necessary to realign the turbine runners to prevent their binding on the speed rings. Every time this was done, it was necessary to deepen the seat for the thrust collar, otherwise it was not possible to line the runner with the speed ring.

About 1929 the conclusion was reached that the concrete in the power house must be growing. Level measurements on a series of bench marks around the power house checked this conclusion. Reference points were also installed across the cracks in the pit liners and measurements of the crack opening were made with an extensometer.

The other concrete structures of the development exhibited no definite signs of volumetric growth. The downstream face of the concrete spillway section and some of the spillway piers had spalled to a considerable extent, but this was attributed to the ordinary deficiencies sometimes found in concrete structures.

By October of 1931 practically all of the guide vanes in the speed ring of unit No. 1 had broken, and it was decided to sever the few remaining guide vanes which had not up to that time shown any signs of cracking. It was thought at the time that this would give the unit a chance to readjust itself to the changed conditions and eliminate for some time the recurring trouble of the runner binding on the sides of the speed ring. As each vane was cut through, the sudden release of energy caused a loud report, stated to sound "as though a gun had been discharged within the scroll case." Inspection of the speed rings of the other two units about this same time showed very little cracking in any of the guide vanes but it was noticed that the crack extending around the pit liner in each of these units was much larger than the similar crack in unit No. 1, where the speed-ring guide vanes had cracked so badly.

R. J. Holden, professor of geology at Virginia Polytechnic Institute, deserves great credit for his appraisal of the action of the concrete as early as November 1935. After studying the concrete and the aggregate petrographically and otherwise, he stated: "certain constituents

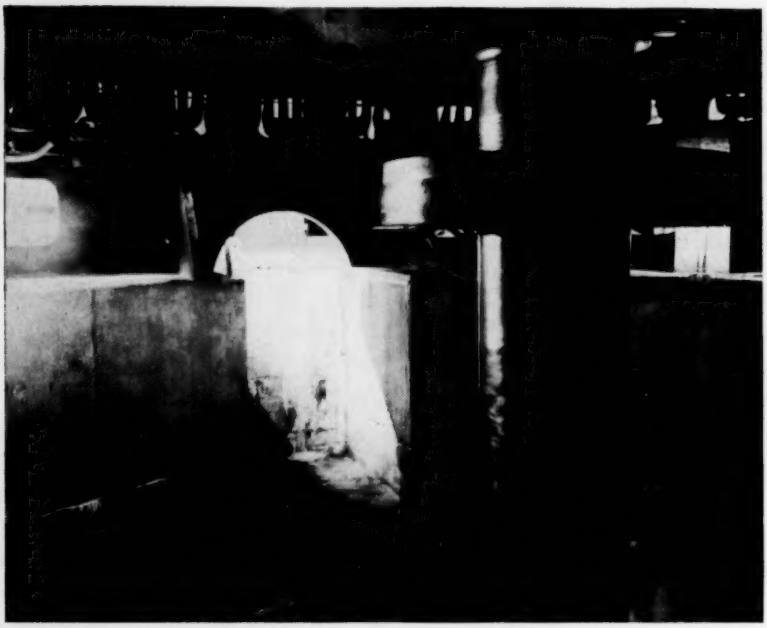

FIG. 2—PIT OF GENERATOR NO. 3 OF BUCK HYDROELECTRIC PLANT. SHOWING LARGE CRACK IN CONCRETE

of the slate (coarse) aggregate had united with certain constituents of the cement to produce new compounds which increased the volume of the concrete and weakened it."

By 1937 the cracks around the pit liner of units 2 and 3 had become so wide and the breaks in the guide vanes of unit No. 1 had increased to such an extent that the need for substantial repairs was indicated (see Fig. 2). The engineers in charge of the work decided, however, that reconstruction could be accomplished correctly only by determining first the true cause of the trouble and the probability of further growth.

A first series of tests was begun at Massachusetts Institute of Technology in 1938 on, (1) cores drilled from the Buck Powerhouse, (2) cores drilled from the phyllite rock used for aggregate, and (3) new concrete specimens fabricated from the phyllite aggregate.

When the concrete cores were drilled at varying distances from each turbine pit, it was noted that the width of the crack diminished as the distance from the pit increased. This might be taken to indicate that the concrete immediately around the pit had not expanded, perhaps

due to leaching, but that concrete some distance away from the surface exposed to the running water had expanded, thus accounting for the cracks.

The first series of tests yielded information of value but did not explain the expansion. The results of these first tests can be enumerated briefly as follows:

CORES

1. The compressive strength of 6 by 12-in. cores of the 26-year old concrete was 2100 p.s.i.

2. The concrete cores stored in water expanded gradually in direct accordance with the temperature; at 74° F. the expansion was about .017 per cent in 3 months; at 150° F. the expansion, exclusive of thermal expansion was .032 per cent in 3 months; and at 190° F., the expansion was .042 per cent in only two weeks.

3. The expansion of rock cores in water at 150° F. was .005 per cent in 3 months; drying caused corresponding, slight contraction.

4. Thermal expansion coefficients for the concrete and rock cores were equal, 4.7 millionths per degree F.

NEW CONCRETE

5. New concrete cylinders containing Buck aggregate and modern cement stored moist at 70° F. remained practically constant in length, while those stored at 50 per cent humidity contracted .098 per cent, which is not abnormal for the high W/c employed.

6. Resistance to freezing and thawing of the new concrete cylinders was very poor; disintegration began in 10 cycles.

7. New concrete cylinders stored in a solution of ½ per cent each of NaOH and KOH at 70° F. remained constant in length for several months. (Alkali action was suspected at this time, but not understood.)

8. New concrete cylinders subjected to alternating temperatures of 70° F. and 150° F. expanded .03 per cent in 32 cycles (even sound concrete of high W/c will expand about this amount under such treatment).

The net results of the first series of tests were that the 26-year old concrete was shown to have potential growth yet remaining and that the aggregate itself appeared to be sound. A tentative conclusion was then reached that the cement must have been unsound, and this seemed to be supported by finding large grains of unhydrated cement and lime in the concrete by petrographic studies, not reported here.

The tentative conclusion of unsound cement was reached reluctantly, because it was inconsistent with another known fact. The concrete in another power plant, constructed at the same time only three

miles away with the same cement but with a different coarse aggregate, had not expanded. The coarse aggregate in this case was crushed gneiss, which was of a moderately basic variety. The gneiss concrete showed abundant disintegration and could only be classed as poor, but nevertheless, there were no clear signs of general expansion.

When T. E. Stanton[1] revealed his findings of concrete expansion due to reaction between the alkalis in the cement and some constituent in certain aggregates, renewed interest was directed to the Buck concrete. A second series of tests was begun in which the chief purpose was to determine whether or not the Buck aggregate would cause expansion when combined with a high-alkali cement.

Length changes were observed on mortar specimens containing the Buck aggregate combined with cements of high and low alkali contents, and stored both at 70° and 110° F. Among the specimens were some to which 1 per cent each of sodium and potassium hydroxide was added to the low-alkali cement. The aggregate was well graded (75 per cent gradation) from No. 14 sieve down, including 15 per cent passing the No. 100 sieve. The mix was 1:2.5 and the W/c was 0.57 by weight.

The results showing the effect of alkalis in the cement are presented in Table 2. At the higher temperature of 110° F. an accelerated expansion occurred that seemed to be related to the alkali content of the cement. For the low-alkali cement, the expansion was slight, for the high-alkali cement an appreciable expansion began at about 8 months, while for the 2 per cent of added alkalis, the expansion began at 6 months or less and was sufficient to cause cracking of the specimens in 8 months.

The specimens at 70° F., even with added alkalis, expanded only slightly up to the last observation at 1 year. This is not surprising, because the concrete in service at about this temperature did not show expansion for many years. It should be pointed out, however, that the laboratory specimens contained 15 per cent of the phyllite aggregate passing the No. 100 sieve, while in the structure, only the coarse aggregate was phyllite. The tests are being continued indefinitely.

Examination of the 29-year old concrete revealed that there had been a chemical attack on the phyllite aggregate. The particles were altered at the surfaces and gelatinous reaction products were visible. Fig. 3 is a good illustration of the peripheral alteration of a phyllite particle. Some of the reaction products are also visible.

[1] T. E. Stanton, Expansion of Concrete through Reaction between Cement and Aggregate, Proceedings A. S. C. E., Dec. 1940.

FIG. 3—AN UNPOLISHED SECTION OF BUCK CONCRETE SHOWING ALTERED BOUNDARY OF PHYLLITE PARTICLE AND REACTION PRODUCTS

The evidence indicates quite convincingly that the Buck concrete expanded due to an alkali-aggregate reaction, probably to form sodium or potassium silicate. It is further indicated that the expansion was of the type which occurs only when a high-alkali cement is combined with an unusual aggregate, in which the silica, or possibly some other constituent, is in reactive form.

TABLE 1—CHEMICAL ANALYSES OF PHYLLITE AGGREGATE USED IN BUCK HYDROELECTRIC PLANT

Silica (SiO_2)	60.9
Iron Oxide (Fe_2O_3)	7.4
Aluminum Oxide (Al_2O_3)	19.1
Calcium Oxide (CaO)	1.1
Magnesium Oxide (MgO)	1.6
Alkalis (as Na_2O)	4.1
Loss on Ignition	3.8

TABLE 2—LENGTH CHANGES OF MORTAR CYLINDERS CONTAINING PHYLLITE AGGREGATE FROM BUCK POWER PLANT

Nature of Cement	Length Change, Per Cent					
	Sealed at 70° F.			Sealed at 110° F.		
	3 Mo.	6 Mo.	1 Yr.	3 Mo.	6 Mo.	1 Yr.
High Alkali	+.006	+.007	+.012	+.009	+.012	+.017
Low Alkali	+.001	−.002	+.006	+.001	+.005	+.007
Low Alkali + 1% NaOH and 1% KOH	0	+.004	+.022	+.017	+.087	+.140

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Evidence in Washington of Deterioration of Concrete Through Reactions between Aggregates and High-Alkali Cements*

By Bailey Tremper†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Deterioration of concrete in two areas in Washington is described. Unsatisfactory results are connected definitely with combinations of certain aggregates and cements. The aggregates when used with other cements and the cements when used with other aggregates have produced concrete of satisfactory durability. Evidence indicates that, with these aggregates, the cements that were used in durable concrete were low in alkali and those used in concrete which has deteriorated were high in alkali. Investigations have not been completed, however, and there is some evidence of a negative nature.

The State of Washington is fortunate in possessing abundant resources of strong, durable aggregates and modern cement factories of more than ample capacity for normal consumption. Concrete, generally speaking, has endured remarkably well notwithstanding severe climatic conditions in Eastern Washington and in the mountainous regions.

There are, however, a few examples of unsatisfactory performance (a very minor percentage of the whole) which taken by themselves make up a volume and an investment of considerable magnitude. Maintenance costs have been unduly high in some instances and the necessity of repairs to additional concrete is in prospect.

The Department of Highways is currently investigating possible causes of unsatisfactory concrete pavements and structures that are

*Received by the Institute April 21, 1941.
†Materials Engineer, State of Washington, Department of Highways, Olympia.

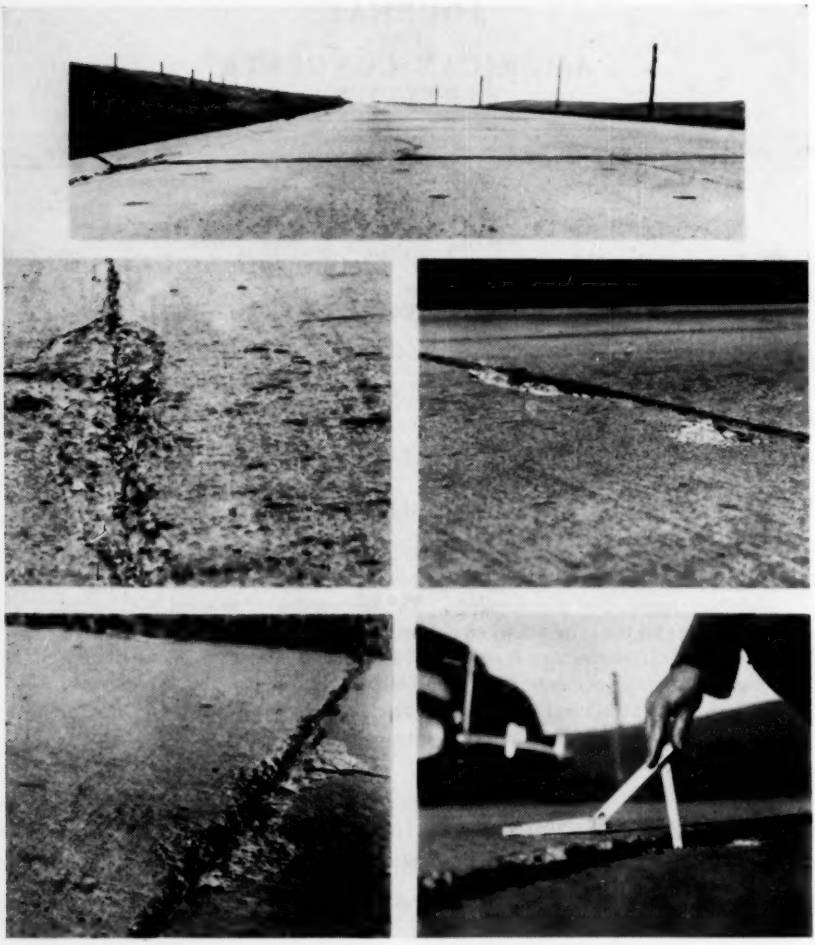

FIG. 1—OFFSETTING AND SPALLING AT EXPANSION JOINTS IN PAVEMENTS. CONDITION EIGHT YEARS AFTER CONSTRUCTION

part of the State Highway system. In these studies, the possibility that cements of high alkali content have contributed to deterioration has been considered. It is the purpose of this paper to describe the nature of the defects observed, the characteristics of the materials used and to present both positive and negative evidence of reactions between aggregates and cements of high alkali content. Conditions as found in two areas within the State will be discussed.

PAVEMENT DETERIORATION

In one of these areas, in the vicinity of Spokane, two types of defect have been observed in concrete pavements. One type has been an abnormal expansion which has exhausted the constructed expansion space of ½ inch in 40 feet and has been sufficient in degree to develop end pressures that have caused severe spalling. A result, possibly more serious than the spalling, has been a vertical offsetting of adjacent slab ends. Offsets up to one inch or more at each expansion joint have occurred over relatively long sections. This condition constituted a decided traffic hazzard particularly since alignment and grades were of a nature to encourage high vehicle speeds. Typical views of such pavement are shown in Fig. 1.

Abnormal expansion has occurred on approximately 13 miles of pavement constituting portions of four separate projects, all constructed during 1930 and 1931. The aggregates for each of these pavements were furnished from the same source. The affected portions of all were constructed with cement from the same factory, designated as A in this report. Three of the projects contain sections in which cement from other mills was used—four in all including cement A. All portions except those containing cement A have been entirely normal in behavior.

The slabs constructed with cement A contain relatively few surface cracks of the type known as map cracks which are freuqently associated with restrained expansion. Vertical offsets in the pavement were promoted by inclination of the expansion joints. The latter was a fault, not overcome entirely at that time, due partly to the type of joint material then in use. Had the joints been more nearly normal to the surface it is to be presumed that offsetting, and possibly spalling at the joints, would not have occurred. Whether or not the expansion under these conditions would have resulted in blow-ups can only be conjectured. The important observation is that neither offsetting nor spalling has occurred under the same conditions with cements other than Brand A.

Notes of the inspectors making condition-surveys of these pavements indicate that tight joints were observed during relatively cool weather two years after construction. There is evidence in the form of mud-jack holes that maintenance men had at some time attempted, unsuccessfully, to correct offsets by raising the slab on the low side of the joints. Such holes may be observed in the top view of Fig. 1. It was not until the spring of 1938, however, that the roughness at the joints became severe enough to constitute a definite hazard to traffic. To remedy the condition, new expansion joint spaces were

cut with air drills. These openings were made at average intervals of 90 feet at a cost of approximately $1000 per mile. This operation has cured, apparently the tendency to offset but the riding quality of the surface has not been restored completely. The pavements show evidence of having been injured structurally since transverse and corner breaks are unduly frequent.

A second type of defect became prominent in pavements constructed with cement A during the year 1932. It consists of the more or less general occurrence of surface cracks which form a random pattern without orientation with respect to edges and joints. The formation is not readily visible unless the observer is facing toward the sun. The width of the cracks is difficult to estimate because of edge-rounding under traffic. Cores indicate that as a rule the depth is less than one inch. Very little incrustation is connected with the cracks at present. Photographs taken a year apart at accurately identified locations do not indicate an increase in width, length or number of cranks during this period. There is no evidence of abnormal expansion of this concrete.

This condition has occurred on approximately nine miles of pavement involving three projects. On two of these, cements from other mills were used. Surface cracking has occurred only on those portions in which cement A was used. The aggregates were from the same source as that furnished for the pavements of abnormal expansion described above.

Cement A was used in a fourth pavement during 1932. Some of the cement was taken from the same storage bin that was used in one of the other pavements. Aggregates were obtained from a different source in this instance and no surface cracking has developed.

Cement from mill A has been used in highway construction since 1921. None of the pavements laid prior to 1930 or since 1932 has developed either of the defects described above although the aggregate involved has been used frequently.

PAVEMENT INVESTIGATIONS

One of the steps taken in investigating possible reasons for the conditions observed was to make a detailed study of operating records at mill A. An interesting discovery was that late in 1929 a certain clay as a raw material for the cement was first used and its use discontinued after 1932. The period of abnormal concrete therefore corresponds exactly with that during which the clay was used in the manufacture of the cement.

The use of this clay did not appear to have resulted in pronounced physical changes in the cement as judged by the standard tests in use

at that time. The evidence indicates, however, that progressive improvements were made in burning the clinker more thoroughly, starting in 1930. Steam tests for soundness were made at exceptionally frequent intervals with this cement both in the highway laboratory and in an independent laboratory. While there were a few rejections for this cause it is not believed that any cement that would not pass the standard test was incorporated in the pavement.

The record of chemical analyses on the other hand points to a difference in composition of the cement as a result of the use of the clay. The sum of the oxides customarily determined in routine analysis (those of silicon, aluminum, iron, calcium and magnesium) plus sulphuric anhydride and loss on ignition was found to be approximately 99.50 per cent as an average prior to the use of the clay. As soon as the clay became a constituent of the raw mix the total dropped to an average value of approximately 98.40 per cent. There is no evidence that changes in methods of analysis were made at this time. The difference in results occurred literally overnight. It indicates an increase of over one per cent in undertermined constituents which in all probability were sodium and potassium oxide.

The indicated content of alkalis did not remain at this high level during the entire period under consideration. During 1931 and 1932 there was a gradual increase in the total obtained in routine analysis probably because of higher burning temperatures and the use of a lower percentage of clay. No record of actual analysis for alkalis in the cement has been found.

As a result of a search made in 1939 three small samples representing shipments of 1931 and 1932 were obtained. These contained from 0.95 to 1.07 per cent of sodium oxide plus potassium oxide. In addition to the direct information yielded by these tests, the results tend to substantiate the conclusion that the cement was exceptionally high in alkalis during the earlier part of the period in which the clay was used.

In 1939 more than 100 cores were taken in the vicinity of Spokane, from pavements of various ages and including both satisfactory pavements and those affected as described above. After sawing the ends and saturating in water, the cores were tested for compressive strength. Those from pavements of abnormal expansion had strengths averaging 84 per cent of those from other pavements of similar age. Cores from surface-cracked pavements had a relative strength of 94 per cent. These results indicate inferior concrete but they do not of themselves show progressive deterioration. No cores had been tested previously and comparisons with construction cylinder tests were not conclusive.

It will be necessary to test cores at a later date to determine the probability of continued retrogression.

After testing for strength, the cores were crushed and sieved progressively, discarding the coarser particles each time, until a fine mixture rich in cement was obtained. After grinding in a pebble mill, the powders were analyzed for water-soluble alkali which was computed as a percentage of the cement. Concrete from expanded or surface cracked pavements was found to contain from 0.27 to 0.53 (average 0.39) per cent of water-soluble alkali (expressed as Na_2O). That from other pavements ranged from 0.11 to 0.25 (average 0.18) per cent except that in three cores containing high-early-strength cement the average was 0.39 per cent. It should be stated that although these determinations were of the water-soluble alkali only, other tests have shown that substantially the entire amount of alkali in cement becomes water-soluble after a period of hydration.

The aggregate used in the defective pavement was a blend from three pits. It consisted of a gravel, a coarse sand and a fine sand. The gravel contains granite, quartzite and chert of low absorption together with minor amounts of impure limestone or calcareous siltstone. The coarse sand is of similar composition. The fine sand contains 2 per cent shale, 26 per cent chert and traces of limestone, the remainder being mainly quartz, quartzite, feldspar and schist.

Bars, 1 x 1 x 10-in., with metal inserts for expansion measurements have been made of 1:2 mortar using the aggregates employed in the pavement and cements of varying alkali content, following the method of Stanton(1). Gravels and coarse sands were crushed to produce fine sands for these tests. The bars have been in storage in sealed containers for periods up to one year. None has shown significant expansion.

Thin sections made from concrete from the pavements have been examined petrographically. Those from expanded slabs show minute fissures and light brown patches of limonite stain to a greater extent than others. The examination shows the results of expansion but is not considered as explaining the cause.

STRUCTURE DETERIORATION

A second area in Washington in which distress is evident in concrete lies along the upper valley of the Cowlitz River southeast of Mt. Rainier in the western part of the state. Here the evidence of chemical reaction between the aggregates and a high-alkali cement seems quite definite. The concrete in question is in a series of highway bridges

[1] "Expansion of Concrete Through Reaction Between Cement and Aggregate," by T. E. Stanton, Proceedings, A. S. C. E., vol. 66, p. 1781 (Dec., 1940).

FIG. 2—HANDRAIL OF CLEAR FORK CREEK BRIDGE. NOTE THAT THE EXPANSION JOINT SPACE IS NEARLY CLOSED

all of which were constructed with aggregates consisting of sand and gravel from bars in the Cowlitz River.

Twelve bridges within a distance of thirty miles were constructed with these aggregates during the period of 1927 to 1937. All of the material came from the same bar except that for the first three bridges which was obtained a few miles down stream.

Each of the bridges has been examined in detail. Seven of the twelve show no outward evidence of distress. Faint signs of crazing are evident occasionally on the handrails but the marks have largely been erased by normal weathering. The cement used in each of the seven bridges was either from mill B or mill C.

The remaining five structures were constructed with cement from mill D. The condition of the concrete varies with age but each has areas in which relatively wide, open cracks appear.

Clear Fork Creek Bridge, a 325-ft. concrete arch, constructed in 1930 was the first of the group in which cement D was used. Open cracks having widths up to $\frac{3}{16}$ in. are prevalent in the handrails and curbs. Along the curb and on top of the handrail these take a longitudinal direction, four to five feet long. On the end posts the cracks are of the random pattern characteristic of expansion where there is no restraint. A knife blade may be inserted into some cracks to a depth of more than two inches. At one point an open expansion joint in the rail (Fig. 2) was found nearly closed in relatively cool weather although there are no signs of spalling at joints. The roadway deck

FIG. 3—CRACKING IN UPPER FACE OF ARCH RIB OF CLEAR FORK CREEK BRIDGE

and sidewalk contain fine cracks but these of themselves would hardly cause concern since they cannot be distinguished with certainty from hair checks that might have occurred during construction. Open cracks appear in the end walls but the columns and the beam and girder system appear unaffected. Longitudinal, open cracks about $\frac{1}{16}$ in. wide appear on the top and bottom faces of the arch ribs at both ends (Fig. 3). Wherever cracking is prevalent in this bridge the concrete gives a dull thud when struck with a hammer. Its appearance may best be described as "chalky."

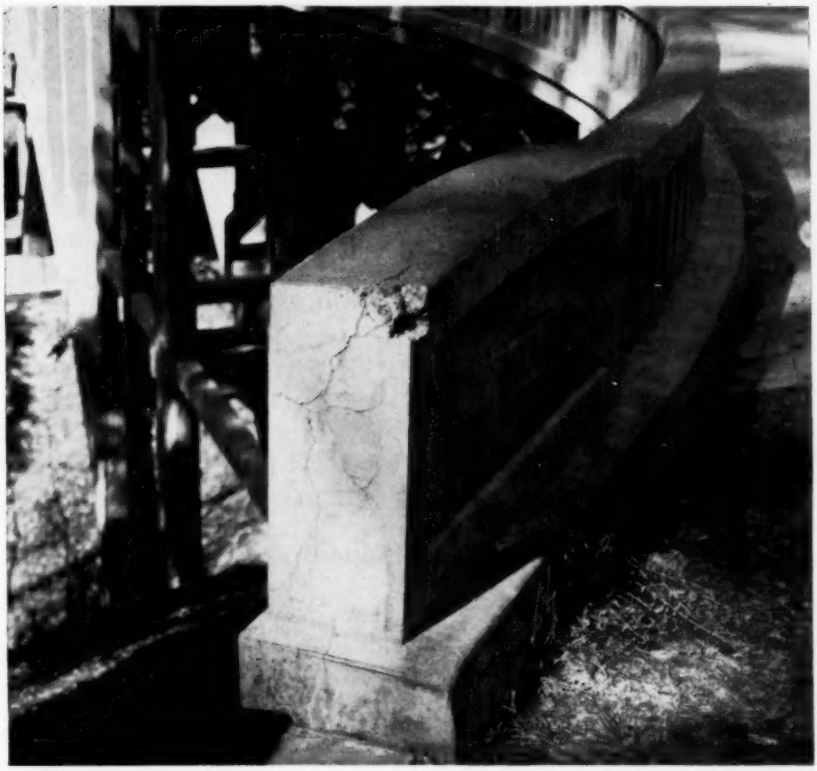

FIG. 4—CORTRIGHT CREEK BRIDGE

Several close-up photographs were taken of this bridge in 1940. A comparison of the same areas a year later indicates that the cracks have not increased in size or number. Some fresh spalling during the last winter indicates that the concrete is less able than formerly to withstand the attacks of weather.

The condition of this bridge is in sharp contrast to that over Purcell Creek which is less than one mile distant and was built a year earlier with brand B cement.

Cortright Creek Bridge (Fig. 4) is a high concrete arch, 200 feet long, with T-beam approaches, constructed in 1933-34. The number and size of open cracks in the handrails (Fig. 5) are similar to those at Clear Fork Bridge. Many of the expansion joint openings in the rails, particularly on the southerly side, are closed or nearly so. Fine cracks appear in the curbs and sidewalks. The deck is free from significant cracking but gives a dull sound in places when struck with

FIG. 5—HANDRAIL OF CORTRIGHT CREEK BRIDGE

a hammer. The arch ribs are difficult of access and have not been examined. Other portions of the sub-structure appear to be in good condition.

Areas photographed in 1940 and examined in March 1941 are seen to have developed new cracks and the older cracks to have increased in length and possibly in width.

Summit Creek Bridge is a concrete arch 199 ft. long constructed in 1935. Cracking in the handrails is less advanced than in the older bridges but expansion joint openings show some closure. There are a few longitudinal cracks along the edge of the sidewalk. The deck contains a few cracks some of which are in pattern form and some longitudinal. Part of the latter are open while others contain hardened gel. Although the arch ribs are not affected, other portions of the sub-structure contain more cracks than the older bridges particularly in columns, beams and arch abutments.

Areas photographed in 1940 and examined in March 1941 are seen to have developed new cracks while the older cracks have increased in length.

Laughing Water Creek Bridge is a continuous girder three-span T-beam structure constructed in 1935. Open cracks appear in the superstructure but they are much less extensive than in the other bridges described.

A half bridge (so designated because it provides only half the roadway on a side-hill location) was constructed in 1937 and is the youngest of the five structures containing cement D. It is of concrete girder construction with a ballast deck and rubble rail. When first examined in 1940, fine cracks were seen on the exposed ends of the girders. In the spring of 1941 the cracks were found to have lengthened and to have increased in number. Nearby is a timber bridge with concrete deck and handrails, built under the same contract. High-early-strength cement from mill B was used in this structure. It shows no cracking or even crazing.

Considering the group of structures built with cement D it appears from the inspections that have been made that cracking does not become apparent until about the third year. Cracking seems to continue for a number of years but may cease when the age of about 10 years is reached. By this time, however, the concrete appears to have become weakened and vulnerable to the effects of weather. These bridges are at elevations of 1300 to 2000 ft. and receive heavy precipitation with frequent cycles of freezing and thawing.

STURCTURE INVESTIGATIONS

Definite information is lacking as to the alkali contents of the cements at the time of use. However, knowledge of the raw materials and of operating conditions at the three mills that have supplied cements for these structures tends to indicate that the alkali contents at the time of use in the bridges were of the same order as has been found in recent analyses. The following determinations were made on cement manufactured in 1940:

Cement	Na_2O	K_2O	Sum
B	0.30	0.19	0.49
	0.22	0.08	0.30
B (High-early-strength)	0.06	0.05	0.11
	0.17	0.05	0.22
	0.12	0.03	0.15
C	0.34	0.20	0.54
	0.56	0.16	0.76
	0.29	0.17	0.46
D	0.51	0.56	1.07
	0.76	0.33	1.09
	0.78	0.29	1.07

A number of laboratory investigations have been initiated which it is hoped will afford results tending to establish more definitely whether or not the observed deterioration has been caused by a reaction between alkalis in the cement and the aggregates. To date, only a few of the experiments have progressed sufficiently to yield significant results.

The Cowlitz River is fed by a glacier on Mt. Rainier. Entering the river above the point at which aggregates were obtained for the bridges under discussion are a number of tributary streams which have their sources in the Cascade range. The gravel used in these structures contains about 33 per cent andesite, 50 per cent lavas which have been altered by agencies such as steam, 6 per cent granites and siliceous tuffs, 9 per cent metamorphics and 1 to 2 per cent durable sandstone. The gravel is sound structurally. The only pop-outs observed in the bridges have been caused by pieces of wood. The composition of the sand is similar.

Bars, 1 x 1 x 10-in., with metal inserts for expansion measurements were made of 1:2 mortar using the sand from five locations in the river bar. These were made with a cement containing 0.63 per cent Na_2O and 0.46 per cent K_2O. The bars have been stored at 70° for 6 months in sealed containers containing a small amount of water. These bars have shown a rather small but continuous expansion with no indication to date of a decrease in rate. Results are given in the following table, each value being the average of 4 bars:

Sample No.	Expansion in Millionths at Age Shown					
	1 Mo.	2 Mo.	3 Mo.	4 Mo.	5 Mo.	6 Mo.
1	100	105	150	175	190	235
1A	90	95	125	125	140	175
2	90	100	135	155	175	210
3	85	95	145	165	200	240
4	105	110	145	160	180	220
5	120	130	175	190	210	260

Stanton noted that mortar containing reactive siliceous magnesian limestone and a high-alkali cement after aging for some time became fluorescent in ultra-violet light, although the original mineral did not fluoresce. Fragments broken from these of the Cowlitz River structures in which cement D was used fluoresce strongly under ultra-violet light. Fluorescent areas predominate over the fractured faces, the effect being strongest in sockets where the gravel has broken away from the mortar. The original outer faces of these fragments appear nearly black. The fragments were sawed and the plain faces were examined under the ultra-violet light. Bright colored rims appear surrounding many aggregate particles and some areas of mortar are fluorescent.

In contrast to this behavior, a fragment broken from Purcell Creek Bridge, in which cement B was used and which is in excellent condition, exhibits only the faintest signs of fluorescence.

The ultra-violet light promises to be a rapid means of detecting active rock types. Cylinders containing Cowlitz River aggregate were made with seven cements of varying alkali content. They were sawed into sections when 7 days old. At this early age fluorescent rims were observed surrounding many coarse aggregate particles. The rims were brighter and more numerous in specimens containing cements of the higher alkali content. Specimens of Cowlitz River gravel were sawed to produce one plain face. They were then coated with cement paste on all but the sawed face. Seven days later fluorescent rims were observed surrounding some specimens. The active aggregate particles were found to be certain of the andesites and certain of the other lavas. The degree of fluorescence increases with age of moist curing but the tests have not been under way for sufficient time to warrant a rigid classification into active and non-active types. None of the Cowlitz River particles is of itself fluorescent.

SUMMARY

The condition of defective concrete as found in two localities in Washington has been described. Deterioration has occurred only when cement from certain mills has been used. When cement from other mills was used with aggregates from the same sources, concrete made under closely similar circumstances has shown excellent durability. On the other hand the cement by itself cannot be held solely responsible for the difficulties because each has been used with good results with other aggregates. The conclusion must be reached therefore that reactions between particular cements and particular aggregates have caused the impairment. While possibly not fully conclusive, investigations indicate that the difference between satisfactory and unsatisfactory durability is closely related to the content of alkalis in the cement. Evidence pointing to such a conclusion is summarized below.

(1) With respect to pavements in the vicinity of Spokane, samples of cement produced at mill A during the period of construction of unsatisfactory concrete were found to contain alkalis in amounts of about 1 per cent. It is indicated indirectly that part of the production from this mill contained alkalis in amounts considerably greater than 1 per cent.

(2) The aggregates used in these pavements contain considerable chert together with small amounts of shale, sandstones and impure limestones. Certain rocks of these types were found by Stanton to react unfavorably with high-alkali cement.

(3) Concrete in the affected pavements contains water-soluble alkali in amounts that are substantially greater than in normal pavements of comparable history.

(4) With respect to structures in the Cowlitz River region, satisfactory results were obtained with cement from mills from which recent production is relatively low in alkalis. Conversely, unsatisfactory performance has resulted in every instance when the cement was from a mill, the recent production from which is relatively high in alkalis.

(5) Mortar bars containing Cowlitz River sand and a high-alkali cement have exhibited a mild but continuous expansion in the laboratory.

(6) Fragments from bridges which have developed open cracks, all containing cement from one mill, fluoresce strongly under ultra-violet light. A fragment from a bridge which is in excellent condition, and contains cement from a different mill, does not fluoresce. In laboratory concrete containing Cowlitz River aggregate specimens containing cements of the higher alkali contents fluoresce in greater degree.

Evidence that such a conclusion is not fully justified may be stated as follows:

(1) Although it is known that cement A at the time of use contained 1 per cent or more of alkalis it has not been established that other cements which gave satisfactory results in the pavements did not also contain alkalis in relatively high percentages.

(2) In one section of satisfactory pavement the present content of water-soluble alkali is as high as the average found in deteriorated pavement constructed with cement A.

(3) It has not been possible to produce significant expansion in the laboratory with mortars containing the aggregates used in the defective pavements and any of several cements of relatively high alkali content.

(4) Although deterioration of the concrete seems well connected with differences in alkali content of the cements used, particularly in the group of structures made with Cowlitz River aggregates, investigations have not as yet entirely eliminated the possibility that other constituents of the cements were contributing factors.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL Supplement concluding this Proceedings volume

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The Nature of the Processes Leading to the Disintegration of Concrete, with Special Reference to Excess Alkalies*

By Charles P. Berkey†

FOREWORD

Dr. Berkey's paper was presented to a conference group at the Bureau of Reclamation, Denver, and his consent given to its publication by the Institute as of possible assistance, through the viewpoint of a petrologist, in an approach to the problems presented by chemical activity of some rocks in the presence of high-alkali cement. It is not to be read as an *explanation* of disturbing phenomena recently observed in concrete structures, but, as the author says, "a suggestion based on petrologic analogy."—Editor

Concrete is essentially an artificial rock. Both in its outer and its internal form it resembles a conglomerate. Its internal microstructures resemble those that characterize incipient metamorphism in rocks, primarily because of the reorganization of original unstable constituents through the hydration processes. Its final stages of transformation from a physiochemical system of water, cement and aggregate to a petrographic product of hardened concrete remind one of certain well-known end-stage changes that commonly take place in the forming of igneous rocks.

The first of these resemblances, that of the conglomerate aspect, is the simplest and most superficial. This appearance arises from the fact that in the average case concrete consists of an "aggregate" of pebbles and sand which finally becomes a more or less substantial mass through the development of a fine binding substance to which nearly its whole strength and stability are due. This is a purely physical aspect that may or may not have further significance, depending upon what the binding substance is and how it came to be where it is.

*Received by the Institute March 12, 1941.
†Consulting Geologist, Columbia University, New York.

In the simplest varieties of such rocks the binding matters have been largely carried in, in solution, by circulating waters and have been deposited by precipitation at the points of contact between the discrete particles making up the mass. In more complicated cases, the fine-grained earthy substances originally deposited inter-stitially between the larger fragments take part also to some extent in the making of a binding material. In such instances the binding material is essentially a reorganization product involving a new process that is more complicated than is direct precipitation out of solution, and more complicated even than precipitation dependent on chemical reaction with original materials. Entirely new products are being made out of simpler originals. Earthy and clayey substances are transformed into definite minerals, which, in the course of their development, attach themselves to the adjacent stable particles and thus tie the whole mass into a more or less firm body. This is the step referred to as *incipient metamorphism*. Metamorphism is commonly carried much further simply by continuing the process over a much longer period, by subjecting the whole system to greater pressure and higher temperature, by introducing dynamic conditions, or by a combination of these factors.

In the more complicated cases there is still closer resemblance to concrete than in the simpler type previously described. But the complexities introduced in the case of concrete take on yet other features, some of which remind one of the crystallization of an igneous magma, especially in the opportunity for end-stage exchanges of elements and mineral replacements and the destructive attack known to accompany the presence of surplus roving "mineralizers" that have not become fixed by incorporation in the new minerals or other stable products.

The roving mineralizers are surplus substances usually volatile or liquid, that do not enter into the making of normal rock minerals. This action is familiar to petrologists acquainted with the life histories of igneous rocks. A magma is essentially a hot rock liquid. Chemically, it is a solution of many substances in many others out of which, under favorable circumstances, a mineral aggregate known as an igneous rock is formed. The usual favorable circumstance is cooling, but formation of the rock may also be brought about by change of pressure or loss of valatile constituents or even by the invasion of precipitating agents. It appears that most magmas have carried extra matters which are essentially surplus stock. Therefore, as the mass cools down and takes an crystalline internal structural form, these substances tend to be forced out and most of them finally escape.

On their way, however, they commonly cause much damage to the structures already built up, such as the common rock minerals that belong there by right. These destructive effects take many different forms. In many instances the original primary minerals become altered in part or in whole by these free-lance elements. Sometimes the original minerals are replaced by entirely different kinds of mineral substances and often, as a result, the rock structure is weakened materially. Occasionally there is complete softening or disintegration.

None of these steps is rare. The nature of the processes is a matter of interpretation, to be sure, but in all cases the basis of interpretation is factual data resulting from direct observation of modifications produced in definite minerals and mineral aggregates known as rock. There can be no doubt of the effects, or the causative agents, or the manner by which the changes came about; the end products are observable and the manner of their formation can be deduced.

The continued hydration of cement in concrete normally results in the ultimate development of a relatively stable product. But changes comparable to those in rocks may also occur. Extra substances may also be present in concrete which, like mineralizers in rock magma, promote a whole series of reactions and reorganizations. In such a situation, destructive attack and reaction by roving elementary substances is to be expected. The principle is not by any means confined to cooling igneous magmas and the resultant igneous rocks; it is merely more apparent because the geologic phenomena have operated over a longer time.

The complexities of the cement itself are sufficient in the average case to initiate a whole train of chemical and petrologic processes and events. And when, as sometimes happens, the "aggregate" joins in the changes that take place by contributing to the reactions, a still more complex situation is developed. Crystallization on a minor scale, exchanges of elementary constituents, reorganization of fine unstable material, and separation of the solidifying and crystallizing elements from the surplus residuary solution take place. This residuary concentrated solution carries all the roving elements not used in the new crystalline set-up. If these elements are chemically active, they may, in their concentrated form, attack even the micro-crystalline material on which the binding and all the strength of concrete depend. If they do that, the whole matrix becomes more or less affected. Its binding structure is destructively attacked, the mass loses strength, and it is on the way to disintegration, just as rocks are that have faced the same genetic history. It may be that the dull "sugary appearance" of broken defective concrete is due to just this process.

Occasionally the problem is still more complex. If the cement is the principal source of the attacking substance, the program of destruction may be carried on indefinitely as the roving mineralizers are released by continued hydration. In that case the results are cumulative and progressive.

The process as I see it is essentially chemical in nature with all the complexities of reaction and reorganization that unstable materials and still more unstable solutions may produce. But the end product is a rock, a "petro-aggregate," normally made up in part of original unchanged mineral substances, and in part of reorganized materials that have been made out of unstable substances purposely added to the mass in the form of cement, and in final part of entirely new crystalline substances precipitated out of solution. Whatever else is present is not needed and in most instances has no business being there, for such substances are likely to be destructive to concrete just as surplus mineralizers are to natural rocks. To my best belief, surplus alkali in cement is of this nature. It seems not to be needed in the building of any of the new products. It becomes a rover, causing progressive disintegration, by reacting destructively with the aggregate and with some of the substances formed in the setting process.

To provide against such destructive attack the active roving elements in the end-stage set must be eliminated. The best way to do that is not to put them in.

Discussion of this paper should reach the A. C. I. Secretary in triplicate by September 1, for publication in the JOURNAL *Supplement concluding this Proceedings volume*

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Discussion of a Paper by Charles P. Berkey:

The Nature of the Processes Leading to the Disintegration of Concrete, with Special Reference to Excess Alkalies*

BY GEORGE L. KALOUSEK†

The paper on the "analogy between rock and concrete" is an interesting generalization. However, the casting of Na_2O and K_2O into too general a role may be misleading in attempted interpretations of the causes underlying the failure of concrete. For explaining these processes it appears desirable that all constituents be kept in mind as illustrated by the following considerations.

Hydrous silica and "water glasses" have been observed [1][2][3] as reaction products of alkalies and aggregates, and the possibility that these products might cause expansion has been advanced. It is known that when water reacts with cement, $Ca(OH)_2$ is split off from the $3CaO.SiO_2$, and a calcium silicate hydrate together with the $Ca(OH)_2$ are precipitated. Solutions of NaOH and KOH dissolve silica and some silicates, both natural and artificial (laboratory glassware, for example); solutions of $Ca(OH)_2$ do likewise except that the attack is slower. Because of this it is probable that SiO_2 dissolved from rocks combines with the available $Ca(OH)_2$‡ to form a calcium silicate hydrate. The formation of the latter might cause expansion of the concrete.

Although compounds of Na_2O and K_2O are generally highly soluble, the possibility that a portion of the Na_2O or K_2O combines as a slightly soluble complex silicate must not be precluded. Thus, Forsen[4] has shown that $3CaO.SiO_2$ placed in solutions of KOH yielded solid reaction products containing about 5 per cent of K_2O. The removal

*ACI JOURNAL, June 1941; *Proceedings* Vo. 37, p. 689.
†National Bureau of Standards, Washington, D. C.
‡If it is assumed that $2CaO . SiO_2 \times H_2O$ is the product formed upon complete hydrolysis of the $3CaO . SiO_2$ in concrete, each lb. of this compound yields 0.32 lb. of free $Ca(OH)_2$.

of a portion of the Na_2O and K_2O from solution as a complex silicate, or other compounds, is important because it implies a limiting concentration of NaOH and KOH. However, if these two constituents are not partially fixed, their ultimate concentration would greatly increase and conceivably might approach saturation values because the mixing water, especially in dry climates, evaporates to a large extent.

Meissner[2] in considering feldspars as unstable aggregates in concrete pointed out that albite might be converted to a zeolite and hydrous silica. Assuming that this reaction occurs in hydrating cements, the possibility of a further reaction of the resulting zeolite with lime with the liberation of alkali hydroxide presents itself with the interesting ramification that the aggregate might be the source of the alkali hydroxide. This is especially true when it is recalled that zeolites are present in certain instances with the feldspars in nature.

The fact that CaO-Al_2O^3-SiO_2-H_2O compounds[5][6] are formed, together with the observation that puzzolanic materials when added to cements may decrease the deleterious effect of NaOH and KOH, evince further the need of investigating Al_2O^3 bearing compounds and the effect of NaOH and KOH on them.

Siliceous magnesium limestone, as shown by Stanton[1] is subject to "alkali attack." The presence of carbonate (CO^3) bearing rocks in concrete is interesting because calcium carbonatoaluminates[7] may be formed. These compounds are analogous[7] to the sulfoaluminates, the formation of which is generally believed to cause the disintegration of concrete in sulfate bearing waters. The formation of carbonatoaluminates merits further study, especially in the presence of NaOH and KOH.

A recent study of a portion of the system CaO-Al_2O_3-SO_3-Na_2O-H_2O at this Bureau[8] indicated that the stable sulfate bearing compounds (in the presence of NaOH) are members of a solid solution series having $3CaO.Al_2O_3.CaSO_4.12H_2O$ and $3CaO.Al_2O_3.Ca(OH)_2.12H_2O$ as end members. Jones[9] in an extensive study has concluded that only $3CaO.Al_2O_3.3CaSO_4.32H_2O$, the trisulfate, is the stable product in the absence of alkali hydroxide. Consideration of the above indicates that if alkalies are leached out from a concrete the sulfate-poor compounds convert to the trisulfate. Since such a conversion is accompanied by a positive volume change, the expansion of concretes under certain conditions of exposure may actually be caused by removal of alkali from the concrete.

The preceding statements, based in part on experimental observations and in part on theoretical considerations illustrate that the mechanism of the expansion of concrete is as complicated as the setting and hardening phenomena. For this reason the need of fundamental research as to the ultimate products of reaction for conditions simulating the normal exposure of concrete is definitely indicated. Phase-equilibria studies will show the end products of reaction. But, since hydrating cements in normal service may only approximate a state of equilibrium, the problem resolves itself into one of rate of reaction between the constituents of cement and aggregate. However, before such rates of reaction can be expressed, it first is necessary to ascertain approximately the conditions prevailing in hydrating cements. Are the products found in the absence of NaOH and KOH widely different from those formed in the presence of a 5, or a 15, or a 50 per cent solution of these two constituents? To answer this and similar questions, investigations of portions of the systems involving the various constituents of cement plus water are needed. For example, a study of a portion of the system CaO-SiO_2-Na_2O (and K_2O)-H_2O as applicable to hydrating cement would show whether lime-poor calcium silicates are formed, or alkali-lime-silica combinations are precipitated. Such knowledge would furnish a fundamental basis for testing the relative stability of aggregate-cement combinations; and also would be greatly useful in elucidating the processes of setting and hardening of cements.

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information. No formalities; just write a letter to "the Editor."

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Sulfur Mixtures for Capping (37-68)

The subject is one which merits extensive investigation. This account of some rather meager tests would serve a good purpose if it stimulated such investigation or publication of information already developed. The use of sulfur or sulfur mixtures for capping concrete specimens seems to have been generally approved by those who recognize that caps should be hard and flat, and not flow appreciably under load. Its convenience, speed, and the fact that it does not affect materially the moisture content of the specimen are decidedly in its favor. However, some experience in our laboratory indicates that the smooth, glossy, and seemingly hard appearance of plain sulfur is somewhat deceptive. It has limitations.

Our initial tests, which prompted the later ones, were made by applying a plain sulfur cap about $\frac{1}{8}$ in. thick to some high strength neat cement cubes. Cubes so capped were compared with uncapped

similar cubes, the bearing faces being ground on a flat plate with carborundum. Similar tests were made also on 3 x 6-in. concrete cylinders, sulfur capped specimens being compared with cement capped ones. Results on sulfur capped specimens are averages of three, and results on ground cubes and cement capped cylinders are averages of six.

Age of Capping Hours	Sulfur Capped Cubes	Sulfur Capped Cylinder
¼	6860	5450
½	6580	5370
1	7700	5450
19	9110	
21		6620

Uncapped Cubes	Cement Capped Cylinders
11700	7210

In addition to the fact that the sulfur was not adequate for specimens of such strength, it was observed that the underside of the cap developed ridges where the cubes cracked under load, suggesting considerable plastic flow. Therefore, a plain sulfur 2x2x4-in. prism, cast and allowed to harden 40 hours, was found to have a modulus of elasticity of about 450,000 p.s.i. This is entirely too low.

To make comparative tests of different mixtures, it was desirable to cast specimens of the mixtures, without resorting to the capping of cement or concrete specimens. This is somewhat of a problem, because sulfur shrinks markedly as it solidifies, leaving pipes or cavities in the interior. Eventually, it was found that fairly dependable cubes could be cast by using a pouring gate of some capacity, completely enclosed in cork as soon as it was filled, in order to retain the heat.

From the more promising indications of preliminary trials, a number of mixtures, including Leadite, were tested. All of these mixtures are fairly liquid, and can be poured with ease.

Compressive Strength at 1 day, p.s.i. (Avgs. of 2)
Proportions by Weight

No.	Leadite	Sulfur	Banding Sand	Fly Ash	Carbon Black	p.s.i.
1	1000	....	....	....	....	7250
2	....	1000	....	....	....	2275
3	....	1000	....	....	20	4760
4	....	1000	....	....	50	4775
5	....	600	400	....	6	5740
6	....	600	400	....	12	5090
7	....	600	400	....	24	5030
8	....	600	300	100	....	7560
9	....	600	250	150	....	6960
10	....	600	200	200	....	6740
11	....	600	300	100	6	7350

Prisms, 2x2x4-in., were cast from No. 1 and No. 8 and the modulus of elasticity measured at the age of 1 day:

No. 1 — 1,840,000 p.s.i.
No. 8 — 1,725,000 p.s.i.

Time would not permit us to cover the field of possible mixtures, and either No. 1 or No. 8 represents a great gain over plain sulfur. Using mixture No. 8, caps were applied to mortar and neat cement cubes representing a range in strength. Bearing faces were ground on a flat plate with carborundum. The test results follow:

Compressive Strength at 1 Day, p.s.i.

Mortar Cubes		Mortar Cubes		Mortar Cubes		Neat Cement Cubes	
Capped	Not Capped	Capped	Not Capped	Capped	Not Capped	Capped	Not Capped
4050	3675	5000	4700	5550	5300	7850	10750
4000	3550	5100	4750	5450	5500	7850	10250
3975	3700	4900	4450	5350	5450	8150	10250
4010	3640	5000	4630	5450	5420	7950	10420

One would judge from these tests that the No. 8 mixture should not be used for strengths higher than 6000 p.s.i. No explanation is offered as to why, in the lower strength mortar cubes, the capped ones show higher strength than the uncapped ones.

A few extra mortar cubes were available and were used to observe the effect of time. In addition, cubes were cast from the mixture and tested at shorter ages than previously.

Compressive Strength, p.s.i.

Age of Capping	Capped Cubes			Not Capped	Cubes of No. 8 Mixture		
					1 hr.	2 hr.	3 hr.
	1 Hr.	2 Hrs.	4 Hrs.		6300	6825	6825
					6200	6750	6850
	7450	7440	7680	7660	6250	6780	6840

The indication of the foregoing is that, while the material hardens for a number of hours in larger volume, the thin cap may be used in an hour, and that the 6000 lb. limit mentioned before may be lower than necessary.

The banding sand referred to is a grade produced by the Ottawa Silica Co. It is desirable to mix the ingredients together thoroughly in advance of melting.

Our method of capping cylinders is simple. A vertical shaft is set in a machined plate. On this shaft are mounted two pairs of arms having a 90 degree angle, to guide the cylinder. A ring slightly larger than the cylinder is set on the plate, a quantity of capping mixture is poured in it, and the cylinder is placed on the capping. The weight of the cylinder forces the excess fluid out over the ring.

Capping concrete blocks is done in much the same way. The block is set on a machined plate, and ½ in. square bars are placed around

it to form a rectangle slightly larger than the block. The block is lifted out, and sufficient capping is poured into the rectangle. The block is then set on the capping, the excess being forced up into the core holes and out over the bars. Quickness in setting the block after pouring is essential.

An important contribution to convenience and speed in the method is made by an electric melting pot, well insulated and thermostatically controlled.* Sulfur is fluid at temperatures a little above the melting point, (about 250° F.), but becomes viscous at higher temperatures. The thermostatic control makes it possible to keep the mixture ready for use without over-heating, while, at the same time, a high heat for rapid melting may be used at the start. Incidentally, by avoiding over-heating, fumes are less noticeable.

About 5 lb. of the mixture are used for two caps on an 8x8x16-in. block, more than half of which is easily reclaimed, by knocking off the edges and the portions over the core holes. The reclaimed mixture is remelted to strain out any pieces of concrete, after which it is poured out in sheets to be broken up and re-used.

The average time per block is 3 to 4 minutes, capping both sides. With this speed, specimens can be tested the same day they arrive, even in the afternoon.—H. G. COLLINS†.

Advantage of Square-Twisted Reinforcing Bars? (37-69)

Q—What advantage if any, is there in the use of square-twisted reinforcing bars?

A—To answer that question, I should like to refer to Bulletin 71, Eng. Expt. Station, University of Illinois, "Tests of Bond Between Concrete and Steel," by D. A. Abrams. These tests, published in 1913, showed very clearly the nature of the bond resistance of a twisted square bar. Mr. Abrams was one of the first to point out the importance of not only the average bond stress developed, but also the corresponding amount of slip between the bar and the surrounding concrete.

It is of interest to consider the resistance of such a bar to slip, beyond that due to the adhesion and running friction which is developed by a plain bar. If the bar is free to rotate it tends to "unscrew" itself from the concrete; since the bar is not generally free to rotate, it develops mechanical resistance to motion. The nature of this mechanical bond is seen from an axial longitudinal section of a square twisted bar. The outlines of such section show a series of gently varying

*Sta-Warm Electric Co., Ravenna, Ohio
†Lehigh Portland Cement Co., Allentown, Pa.

scallops or waves; in other words a series of alternate wedging and non-wedging tapers on the surface of the bar. The resulting load-slip curve might be predicted from Abrams' other tests on tapered bars. However, he made plenty of actual tests on the twisted square bars to show that the twisted square bar behaved much like a plain bar up to a slip of 0.01 in., which was usually the point of maximum bond stress for the plain bars. Beyond this slip the bond stress decreased; this was evidently due to the loss of even frictional resistance on the non-wedging surfaces of the bar, combined with only a slight increase in bond on the wedging portions. In these bars, in which the pitch, or length for one full twist, was 12 bar diameters, slips of 0.04 to 0.08 inches were required before the bond regained its original value. Beyond this slip a considerable increase in bond stress was attainable if the concrete did not split. However, a slip of even 0.04 in. is far beyond the amount considered critical in reinforced concrete, since such a slip in a beam might induce premature failure by diagonal tension, or other causes. The twisting and splitting effect of the bar under stress also tends to produce localized failures. Hence, it may be concluded that the twisted square bar has little or no advantage over the plain bar.—F. E. RICHART*

The Navy Department's Fixed Fee Contracts (37-70)

Q—What are the advantages to the public of the Navy form of fixed fee contracts?†

A—Honestly, I have been trying to live that "fixed-fee contract" business down. The fixed fee contract was revived by me in January 1939 for a specific purpose. We had to build air bases in the western Pacific and Alaska under conditions which were almost completely unknown. I mean by that, not natural conditions but conditions of transportation and conditions of resistance of personnel to very trying surroundings. Now, I felt that the way to do this thing was to get a bang-up "doctor" to perform this operation and by a bang-up doctor, I mean a bang-up contractor. The way to get a fellow like that is to pick him yourself, if you have confidence in yourself, give him a fair fee to do a job of work, and the rest is easy. The hard job was to put that over with Congress. The difficulty was that during the World War the cost-plus percentage form of construction contract had come

*Research Professor of Engineering Materials, University of Illinois, as member A. C. I. "Board of Experts" at "Quiz Session" of Institute's, 37th Annual Convention, at which question was asked and answered.

†This question was presented to and answered by Rear Admiral Ben Moreell, Chief Bureau of Yards and Docks, Navy Department, at the "Quiz Session" of the Institute's 37th Annual Convention, Washington, D. C., Feb. 18-20, 1941.

into very wide use and under those high pressure conditions had undoubtedly been abused by some contractors and maladministered, I would say, by some government representatives.

So I began to work up an argument to sell this fixed-fee form of contract to Congress. The advantages, as I pointed out to the committees, are, first, that by this form of contract, if political influences can be avoided (and they can be avoided if you are determined) you can pick for a particularly difficult and hazardous job, one of a number of the first flight of contractors to do the work for you.

It is not generally known outside the government service that under the competitive bidding form of contract, the Comptroller General has established precedents which, in effect, say that a contractor who can provide a bond thereby establishes himself as a responsible bidder. Under the fixed fee form of contract, we negotiate a contract after consultation with at least three contractors who are regularly engaged in work of comparable magnitude and character. You can pick the first flight of contractors. What does this get for you? It assures you that if the job is possible, this fellow will get it done. It establishes a condition under which the government, which is better able to take risks than any other organization in the country, is taking the risk of working under unknown conditions; it enables you to get a quick start—which in these times is of the utmost importance. The start is made by organizing the forces as soon as the contract is let—even before the preliminary plans are ready. In the case of the Western Pacific Island bases, we began to establish a shipping depot at Alameda, just across the Bay from San Francisco, as soon as the contract was awarded. It enables you to make changes very quickly on an equitable basis, because the contractor knows, no matter what change is made, he will be paid for his out-of-pocket costs and if the change is sufficient to constitute a change in the basic scope defined at the time the contract was let, he will get more fee if the change constitutes an increase, or less fee if it is a reduction in the scope. Another thing, it sets up a very close cooperation between the governmental directing forces and the contractor, so that this mutuality of interest, you might call it, can work to effect economies in the ultimate cost.

Now what would happen if we were to attempt to obtain a contractor of the best class to construct an air base, let us say, on an isolated island in the Pacific, where the working conditions were unknown, where there was no way of telling in advance just how much must be furnished in the way of diversion, living facilities, recreation and vacations and all of those factors that go to control the cost of the job?

The way to do that under competitive bidding would be to establish restrictive conditions for qualification, if you were able to get away with it. That would limit the bidding to prudent and capable contractors. If you do that, what is such a contractor going to do? He is going to include in his bid a very large contingent item because he *is* a prudent contractor. Now, as I said before, the government takes the risk under this fixed-fee form of contract and you do not pay for contingencies which may never arise. There is no incentive on the part of the contractor to increase the cost of the work, because by doing so, since it costs money to spend money, he is paying out for overhead which we do not reimburse to the contractor, i. e., he is drawing out the work and by drawing out the work he is also increasing his overhead charges which come out of his fee.

To summarize, then, the advantage of this fixed-fee form of contract is, first, speed in initiation, which means speed in ultimate completion; second, economy; third, flexibility in adapting the structure to the purpose for which it is intended. I think that those three reasons, and particularly the first, the speed reason, amply justify this form of contract. Now don't misunderstand me, gentlemen. Under ordinary conditions, in ordinary times, I am a strong advocate of the competitive bidding form of contract for public works. One reason why I am a strong advocate for competitive bidding is because of what I have gone through since the summer of 1939. Inevitably, in this fixed-fee form of contract, the public official is subjected to the strongest of political and other pressure and it takes a lot of nervous energy and a lot of resistance to combat that and to persist in the determination that the award of fee contracts will be on merit only.

The Bureau of Yards and Docks of the Navy Department has published a comprehensive treatise on the fixed-fee form of contract and this will be mailed to anyone who asks for it.

*Technical Tedium or Otherwise** (37-71)

That paper by R. W. Crum on "Technical Tedium or Otherwise" interests me immensely, and I move its general adoption. The time element is most important and this is well handled, but I like especially the sentence: "The first slide comes on and turning your back to the audience you rather confidentially tell the screen what the picture is about." I only regret that Mr. Crum didn't mention that the slide was upside down.—C. T. Bishop†

*Journal Amer. Concrete Inst., April 1941, p. 589.
†Dept. of Civil Engineering, Yale University.

*Effects of Materials on Cracking Tendency in Dams?** (37-67)

S. L. Meyers' notes on concrete for Conchas Dam, supply the stimulus for the following comments and data on the Gallipolis and Winfield Roller-gate Navigation Dams. The Gallipolis Dam is on the Ohio River, 13 miles below the mouth of the Kanawha, and the Winfield Dam is on the Kanawha River, 31 miles above its mouth.

These two navigation dams involved the placement of approximately 557,000 cu. yds. of concrete and constitute a part of the Kanawha River Recanalization Project, the design and construction of which came under the direction of the Huntington District Office, Ohio River Division, Corps of Engineers, U. S. Army. They, together with two other units of this project, were constructed by the Contracting Division of Dravo Corporation—started in 1934 and completed about three years later.

The maximum thickness in the structures was of the order of 30 ft. and the majority of the lifts varied from 10 to 18 ft. Distance between vertical construction joints averaged about 40 ft.

Since the two navigation dams had practically the same construction schedule, the Gallipolis Dam will be discussed henceforth, as representative of both. Temperature rise data of the concrete were obtained through the use of electrical resistance thermometers. Table 1 shows comparable average characteristics of the cements used and presents, along with other information, average temperature-rise data for the locks and for the dam.

A comparison of temperature-rise data for concrete placed in the locks, where coarse-ground cement was used, and in the dam, where a modified normal-ground cement was used, bring out several points of interest. Analysis of temperature-rise curves, cement data, and concrete-mix designs have made it possible to compare results on a common basis for the two types of cement.

Although the normal cements were higher in 3CS and 3CA (the major heat and early strength producing compounds) they were much coarser ground than were the modified cements—which coarseness would tend materially to reduce their early heats of hydration and strength.

From these considerations it is evident that there is nothing contrary to established concepts in the tensile strength results. The average tensile strength at 28 days for the cements in the dam is 430 p.s.i.,which is only about 15 p.s.i. higher than the average for the

*JOURNAL, Am. Concrete Inst., Feb. 1941, p. 517.

locks. From this it may be seen that the greater proportion of high-strength-giving compounds in the locks is balanced by the greater fineness of the cements in the dam (Table 1), to produce practically equal tensile strength in the two cases.

As pointed out earlier, the cement factor for the dam was a half sack per yard higher than it was for the locks—5.0 sacks for the former and 4.5 sacks for the latter. For purposes of comparison in this discussion, these are called "nominal" cement factors. "Effective" cement factor equals nominal cement factor times the percentage passing the No. 200 sieve, and is used so that results for the locks and for the dam may be compared on a common basis. This method of comparison seems justified as a result of tests in which a quantity of cement larger than 200-mesh was obtained and incorporated in 2 by 4-in. mortar cylinders. A negligible amount of heat was generated at 28 days and the specimens were so weak that they could not be stripped for 7 days, the compressive strength at 28 days and at three months being only 300 p.s.i.

For this reason, and because of the difference in nominal cement factors on the two projects, the so-called effective cement factor is used to reduce the cement factor in both projects to a one-sack mix. Nominal and effective cement factors are given in the table for each brand of cement. Below these values are shown values for average water-cement ratios and compressive strengths for the locks and dam. The compressive strengths and effective water-cement ratios fall on established curves for the relationship between water-cement ratio and compressive strength.

From a study of the temperature rise it will be noted that the dam shows a 3°F. higher rise in temperature. Considering only the fact that more cement was used in the dam, and that it was much more finely ground, it would be expected that this difference in temperature rise would be greater. Keeping in mind, however, the fact that the tri-calcium compounds are considerably higher in the cements used for the locks, the very slight difference is not surprising.

The last step in comparing the temperature rise of concrete in the locks and in the dam is shown at the bottom of Table 1 and in Fig. 1. The total temperature rise for each of the cements is divided by the effective cement factor (number of sacks of cement passing the No. 200 sieve, per cu. yd. of concrete); this gives the temperature rise per effective sack of cement per cu. yd. of concrete. On this basis of comparison, the temperature rise per sack of cement for the locks is approximately 3°F. higher than it was for the dam. This is significant because it indicates that the coarse-ground cement was not effective

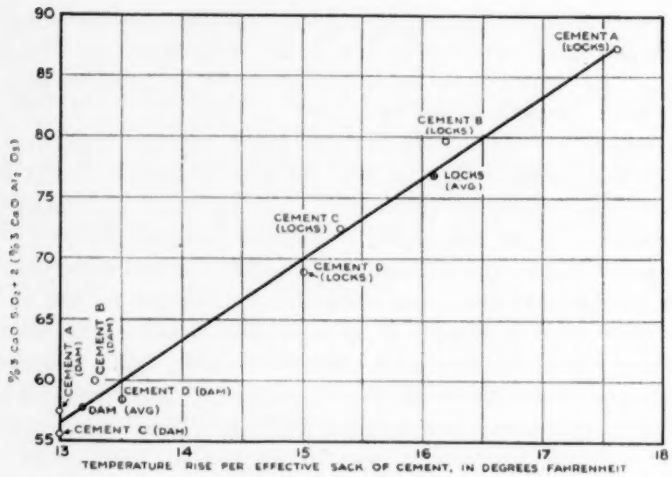

FIG. 1—CORRELATION BETWEEN TEMPERATURE RISE IN CONCRETE AND A FUNCTION OF THE TRI-CALCIUM COMPOUNDS

in reducing the temperature rise in the concrete. If the coarse-ground cement had had a more uniform grading than that used in these projects, the results might have been better—assuming, of course, the same chemical composition.

In Fig. 1, the temperature rise per effective sack of cement is plotted against the tri-calcium compounds. Since it is known that $3CaO.Al_2O_3$ generates approximately twice as much heat at 28 days as $3CaO.SiO_2$ the temperature rise is plotted against the function $3CaO.SiO_2 + 2(3CaO.Al_2O_3)$. The value of this function is considered a good index of heat generation in concrete.

The average maximum temperature (that is, placing-temperature plus temperature rise) reached in the concrete during the summer months was 145° F., and in winter 120° F., which is a normal range under ordinary conditions of placing concrete. The mean annual temperature for this section is around 58° F., and th/ average prevailing ambient temperature 80° F. during the summer months. The time required for the temperature of concrete placed in the summer to drop to the prevailing ambient temperature was about one month, the drop being about 65° F. The drop of approximately 87° F. to the mean annual temperature required more than a year. The time required for the temperature of concrete placed during cold weather to drop the 62 degrees to mean annual temperature was from two to three months.

TABLE 1—RESULTS AND OTHER PERTINENT DATA COVERING TEMPERATURE RISE TESTS CONDUCTED AT THE GALLIPOLIS LOCKS AND DAM

Gallipolis Locks				Type of Information	Gallipolis Dam			
Coarse Ground Cement—Brand					Modified—Normal Ground Cement—Brand			
A	B	C	D		D	B	C	A
57.3%	57.0%	49.3%	51.5%	$3CaO \cdot SiO_2$ (C_3S)	48.7%	47.1%	44.1%	46.1%
14.2%	11.4%	11.4%	8.6%	$3CaO \cdot Al_2O_3$ (C_3A)	4.7%	6.3%	5.6%	5.8%
14.0%	15.1%	23.0%	24.1%	$2CaO \cdot SiO_2$ (C_2S)	25.0%	29.9%	27.3%	28.7%
85.7%	79.8%	72.1%	68.7%	$C_3S + 2C_3A$	58.1%	59.7%	55.3%	57.7%
450 p.s.i.	422 p.s.i.	405 p.s.i.	404 p.s.i.	Tensile strength, 28-day	493 p.s.i.	456 p.s.i.	441 p.s.i.	414 p.s.i.
16.5%	17.2%	17.2%	19.1%	200 Sieve; % retained	2.3%	5.3%	3.8%	8.5%
4.5	4.5	4.5	4.5	Nominal Cement Factor	5.0	5.0	5.0	5.0
3.76	3.72	3.72	3.64	Effective " " (Sacks per cu. yd.)	4.89	4.74	4.82	4.58
1 : 2.62 : 5.58				Average Mix (by weight)	1 : 2.35 : 4.89			
1400 (avg.)				Surface Area—sq. cm./g.	1850 (avg.)			
3.7 sacks/cu. yd.				Average Effective Cement Factor	4.75 sacks/cu. yd.			
0.79				Average Nominal W/C Ratio	0.775			
0.94				Average Effective W/C Ratio	0.815			
4260 p.s.i.				Average Compressive Strength 28 Day	5039 p.s.i.			
66°F.	60°F.	57°F.	54°F.	Temp. Rise—10′ Holes	66°F.	63°F.	62°F.	59°F.
17.6°F.	16.2°F.	15.3°F.	15.0°F.	Temp. Rise per Effective Sack Cement; 10 Foot Holes	13.5°F.	13.3°F.	13.0°F.	13.0°F.

Surface cracks were more severe for concrete placed during warm than during cold weather, probably because of the greater temperature drop from maximum to mean annual. The more pronounced this drop in temperature, of course, the greater the tensile stresses due to contraction. This was closely related to form removal, indicating that schedule of construction plays its role in cracking. The surface cracks developed in the Gallipolis and Winfield lock walls were far more severe than those in the dams. This is believed to be primarily due to the higher percentage of tri-calcium compound content of the cement in the locks. Laboratory investigations have shown that concretes containing lower 3CS and 3CA content exhibit a greater degree of extensibility; that is, they are able to flow more in tension than are similar concretes containing cements high in the tri-calcium compounds.

On the whole, the surface cracks that have appeared on the walls of the structures are not considered serious. They do not affect the structural strength, and climatic conditions are not severe enough to cause serious weathering. Further, indications are that in time calcium carbonate will form and seal the cracks, since the water coming in contact with the structures is not chemically aggressive.—JOHN C. SPRAGUE*

*Development Engineer, Dravo Corporation, Pittsburgh.

How to Avoid Shrinkage in Concrete Masonry Walls? (37-72)

Q—Experts seem to recommend that concrete masonry units be dry when laid as otherwise their shrinkage upon drying is likely to induce vertical cracks. However, during hot, dry, windy weather it is difficult to obtain satisfactory mortar joints when the units are laid in a dry condition. Thus we seem to be confronted with a dilemma. What is the best solution?

A—In the vast majority of cases concrete masonry units should be dry when laid in the wall to reduce volume change to a minimum. Under unusual circumstances where the units are dry and when the weather is hot, dry and windy, it may be desirable to vary construction practices as follows to improve the bond between mortar and the units. Use a wetter, more plastic mortar than ordinary in laying the units. Spread only enough mortar on the bed joint to lay one unit and set it in place immediately. The fact that the mortar is wetter will compensate for some absorption of water by the units. Under extreme cases it may be found desirable to dampen the surfaces of the units over which mortar will be spread. This can be done by drawing over them a paint or whitewash brush which has been dipped in water. Under no circumstances should the units be dipped in water or to be soaked with a garden hose. Mortar ordinarily used in laying concrete masonry units has good water retaining capacity which would tend to prevent loss of moisture to such an extent that it would preclude the formation of a good bond and a strong mortar joint.—W. G. KAISER*

*Manager, Cement Products Bureau, Portland Cement Assn., Chicago.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Concrete deterioration at Parker Dam

R. F. BLANKS, *Engineering News Record*, Vol. 126, No. 13, (Mar. 27, 1941), pp. 46-49. Reviewed by S. J. CHAMBERLIN

A shorter version of the Parker Dam problem described in the A. C. I. JOURNAL, April 1941, by H. S. Meissner.

Large concrete warehouses built with moving falsework

Engineering News Record, Vol. 126, No. 17, (Apr. 24, 1941), pp. 52-55 Reviewed by S. J. CHAMBERLIN

The thin barrel-arched roofs which act as thin concrete beams are carried by heavy reinforced concrete girders spaced 40 ft. apart, which in turn are supported by concrete columns. The roof slabs and girders are constructed on trussed timber forms that are rolled ahead as the work progresses.

Strengthening the frame of a 27-year-old concrete building

C. P. LEWELLYN, *Engineering News Record*, Vol. 126, No. 11, (Mar. 13, 1941) pp. 88-91. Reviewed by S. J. CHAMBERLIN

To increase the load-bearing capacity of the building, columns were incased in concrete, and spans were cut down by steel beams and additional steel columns. Enlarged column caps were placed from the floor above to furnish bearing for new steel girders. Because of ease of placement, hoops with hooks instead of spirals were used to reinforce the new column shells.

Traffic in London, England

HELLMUTH WOLFF, *Die Betonstrasse*, Vol. 15, No. 12, p. 147 (Dec. 1940). Reviewed by A. U. THEUER

This article, primarily intended for German readers, gives a graphic description of pre-war street traffic in London. A number of statistical comparisons are drawn between traffic in London and Berlin. A maximum rate of surface travel in London is stated to be from 11 to 12 miles per hour. The volume of traffic in London is estimated to be three times that of Berlin, although the total population of London is but twice as great.

Prestressed concrete water tanks

Engineering News Record, Vol. 126, No. 11 (March 13, 1941) pp. 91-93 Reviewed by S. J. CHAMBERLIN

The walls are constructed in separate vertical segments or panels placed alternately to simplify forming and to permit each vertical segment to be completed in a single placement of concrete. After seven days a layer of ⅞-in. square bars is put on bearing directly against the outer surface of the concrete. After prestressing a 2½-in. thickness of shotcrete is applied. Following a second seven-day period a second layer of bands around the lower portion of the tank is put on and a 2-in. cover of shotcrete applied.

Soil cement runway for army airbase placed with enrichment in top inch

Engineering News Record, Vol. 126, No. 15, (Apr. 10, 1941), pp. 45-47
Engineering News Record abstract

Low-cost runway construction at the Northeast Army Air Base used ten per cent by volume of portland cement well mixed for a six-inch depth and the surface enriched by additional cement in the top inch. Heavy duty field cultivators and harrows, rotary tillers, sheepsfoot and tandem rollers and a bituminous distributor for application of water constituted the principal equipment required to give a density and weight equal to concrete.

A system of road slab (transition) dowels

KARL SACK, *Die Betonstrasse*, Vol. 15, No. 12, p. 154 (Dec. 1940) Reviewed by A. U. THEUER

A dowel-assembly for expansion joints in road slabs is described. Two rows of trapezoidally-shaped wire stirrups are placed, one on either side of a plank. The stirrups are placed approximately 12 in. on center and each row connected by three longitudinal bars to which they are spot welded. The dowels (28 in. long and 0.87 in. in diameter) are then inserted in holes drilled through the plank and the two rows of stirrups secured at either end of the dowels. The system is assembled in the field with the aid of a special assembly table and placed as a unit at proper intervals in the roadway.

Shear and bond stresses in wedge-shaped reinforced concrete beams

ROBERT B. B. MOORMAN, *Civil Engineering*, Vol. 11, No. 2, p. 114. Reviewed by J. R. SHANK

Professor Moorman derives expressions for shear and bond for wedge-shaped beams. The usual expressions V/bjd and $V/\Sigma_o jd$ are modified by subtracting from V the value, $\frac{M}{d}(\tan \alpha_c + \tan \alpha_t)$, where M is the bending at the point and α_c and α_t are the angles that the compression and tension fibres make respectively with the normals to the section taken at right angles to the neutral axis.

Cantilever forms at Friant Dam

Engineering News Record, Vol. 126, No. 15, (Apr. 10, 1941), pp. 56-57 Reviewed by S. J. CHAMBERLIN

The principal members are 10-ft. timbers set vertically on 9-ft. centers. Three 1¼-in. bolts pass through the timbers, the bottom bolt connects with a threaded anchor in the previously placed concrete, the bolt at mid-height utilizes a connection just above the previously placed concrete, and the top bolt serves as a spacer to which the next succeeding anchorage is attached. Between the lower two bolts is a threaded rod which serves as a jack for adjusting the form to position. Longitudinal

runners support the wood facing which is lined with absorbent material on the upstream and downstream faces. The forms are 50 ft. long and form a complete block for a 5-ft. lift.

Temperature stresses in reinforced concrete chimneys

JOHN A. PIETERS, *Beton und Eisen*, Vol. 39, No. 3, p. 39 (Feb. 5, 1940) Reviewed by A. U. THEUER

In the computation of stresses in large chimneys due to temperature changes, the restraints imposed by and at the base are usually neglected. The horizontal radial stresses at the base reduce to zero; also, the rotations. Secondary stresses, however, it is pointed out, are present and should not be neglected in large structures. An extensive analysis of the stresses at the base of chimneys is given with the following conclusions: A very heavy insulating layer should be included over the base and around the inside of the lower section of the chimney. Another construction detail recommended is the provision for keying the base of the vertical walls to the horizontal foundation slab.

Design and construction features of the Claytor hydro development

PHILIP SPORN and H. A. KAMMER, *Engineering News Record*, Vol. 126, No. 11, (Mar. 13, 1941) pp. 64-68. Reviewed by S. J. CHAMBERLIN

An elaborate program of grouting and foundation consolidation was necessary because of the unusual structure of the underlying limestone which changed to dolomite under each of the river banks. In the absence of sand it was found a satisfactory grout could be obtained by using one part of cement to two parts minus-100 rock fines. All of the coarse and fine aggregate for the concrete structures was manufactured from the dolomitic limestone available at the site. The coarse aggregate was from 3 in. to $\frac{3}{8}$ in., and the fine was crushed so that all of it passed a No. 8 with 15 per cent passing the No. 100 sieve. The gradation was such that the quantity of each successively smaller size was about an equal percentage of the preceding size. 0.85 bbl. of cement per cu. yd. with a maximum water-cement ratio of 0.75 by weight was used for interior mass concrete, and 1.2 bbl. with a water-cement ratio of 0.57 for exterior concrete facing.

Potassium sulfate in cement clinkers

G. L. KALOUSEK, C. H. JUMPER AND J. J. TREGONING. *Rock Products*, Vol. 44, No. 4, Apr. 1941, pp. 52-54. Reviewed by ROY N. YOUNG

Twelve commercial portland cement clinkers having a wide range of K_2O and Na_2O contents were used in this investigation. Each clinker was ground to a fineness of 1800-1900 sq. cm. per gram and made into a paste using 7000 g. ground clinker and 2450 g. of water. After mixing for $2\frac{1}{2}$ minutes the solution was removed by special filter in 2 to 4 minutes. Another set of solutions was made from pastes which had been left standing 1 hr. and 55 min. after mixing. It was found that the amount of K_2O, Na_2O and SO_3 varied greatly in the different solutions. One per cent to 58 per cent of the total K_2O in the clinkers and from less than 1 per cent to 58 per cent Na_2O dissolved in 7 minutes. The rate of solution was much slower between 7 minutes and 2 hours. No correlation could be found between dissolved Na_2O or Na_2O+K_2O and SO_3. However there was correlation between the K_2O and SO_3 among solutions from clinkers which contained readily soluble K_2O. Also a relation was found between the K_2O and SO_3 in the clinkers which contained more than 0.3 per cent K_2O. It was concluded that K_2O in many clinkers is combined partly as K_2SO_4 which is readily soluble.

New system of pre-cast floor and roof construction

Concrete Building & Concrete Products, Vol. 16, No. 3, Mar. 1941, p. 47. Reviewed by J. C. PEARSON

This system is stated to be in use in building many hundreds of houses in Great Britain. It is designed to eliminate temporary form work, for economy in steel and surface finishing, low thermal conductivity and rapidity and ease of erection. The system comprises precast beams made with a high grade reinforced concrete core and a thin casing of foamed slag concrete. The casings are convenient lengths of thin light weight concrete (70 lb. per cu. ft.), L-shaped in cross-section. They are set on a level surface, two L's making a beam form, and as many beams as desired being cast at one time by setting the adjacent forms in contact. The reinforcements are placed in position, and the whole set up poured at one time. U-shaped stirrups are set in the high grade concrete to form a bond for the topping and to afford convenient handling. After the beams are placed, lightweight slabs are laid across them resting on the upper edges of the casing, giving somewhat greater depth to the beams when the topping is poured. As a rule, the casing is cut away at the supports, but it serves as a nailing concrete for ceiling boards. Beams for 12-ft. spans weigh 16 to 17 lb. per ft., and the over-all weight of a finished floor or roof is 37 to 40 lb. per sq. ft. If ceiling boards are used, the thermal conductivity of this type of roof is said to be 0.28 BTU per sq. ft. per hr. per deg.

Bond between concrete and steel

HERBERT J. GILKEY, STEPHEN J. CHAMBERLIN AND ROBERT W. BEAL, Iowa Engineering Experiment Station, Ames, Ia. Bulletin 147. Reviewed by C. V. ARMSTRONG

Contrary to the previously accepted belief, bond resistance between concrete and steel is not proportional to the compressive strength of standard-cured concrete. Neither is the bond developed by an added length of embedment of bar proportional to the added length of embedment.

Their investigation, which was conducted with the cooperation of the Highway Research Board, found that the development of bond resistance is progressive along the bar. At any point the maximum intensity of resistance is obtained and passed as load on the bar increases. Less bond is developed with a smooth bar. Deformed-bar specimens perform in the same manner as plain-bar specimens until slip has been sufficient to bring some of the lugs into bearing.

Loose rust was found to have no significant effect on the bond properties of reinforced steel bars. Vertically-cast beams were found to have stronger bond resistance than horizontally-cast beams.

The complete study, which includes investigation of the effect of length of bar embedment, strength of concrete, type of bar (plain or deformed), rust on bars, and orientation at casting on the bond developed in pull-out, beam, and beam-type specimens, and also includes a comprehensive list of abstracted references, is reported in the 120-page bulletin which may be obtained without charge from the Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa.

Evaluating a puzzolan

H. MCC. LARMOUR, E. L. MCMASTER AND WILLIAM JAQUES. *Rock Products*, Vol. 44, No. 3, Mar. 1941, pp. 52-56 and 87. Reviewed by ROY N. YOUNG

Several natural silicious materials mixed with hydrated lime and with portland cements were studied with reference to puzzolanic action. The extent of the action during any curing treatment is indicated by the change in solubility of the mixture. This is based on the assumption that the puzzolanic effect results from the reaction

of the lime with the material and that insoluble silicates are changed to soluble forms. (Solubility values were obtained by determining insoluble residue, according to A. S. T. M. C114-35T).

It is considered that the maximum degree of reaction can be brought about by treatment in the autoclave. The extent of the reaction under any given curing conditions is expressed as ratio (per cent) increase in solubility found to maximum solubility determined by the autoclave. The autoclave treatment does not indicate the rate of reaction nor gain in strength of mortars under other curing conditions. Neither does the amount of reaction determined at various times and conditions of curing correlate with the corresponding strengths of mortars. Cured at 70°F., standard cement mortar gave the highest strengths, ground Ottawa sand mixtures the lowest, and pumicite in between. At 212°F. the reverse was true.

It is concluded that silicates of low Al_2O_3 content are slowly reactive at normal curing temperatures but highly reactive at elevated temperatures as compared with silicious materials containing relatively high R_2O_3 contents. Increasing the fineness of the puzzolan increases the rate of reaction.

Production and use of pumpcrete

KAYSER, *Beton und Eisen*, Vol. 39, No. 3, p. 33, Feb. 5, 1940 Reviewed by A. U. THEUER

A short review of the development of concrete placement methods is given by way of introduction. Pumpcrete, from a pump which forces concrete through a pipe line to its final point of delivery, is then described in detail. The following points are emphasized with regard to the successful use of this latter method: The freshly mixed concrete must exhibit a pasty-plastic water-retentive character. Sufficient fines are essential. The following aggregate distribution recommended by one of the pump manufacturers is given:

Fine aggregate:	0 to 0.24 mm at least 5 per cent 0 to 0.5 " at least 10 per cent 0 to 1.0 " at least 20 per cent 0 to 3.0 " at least 35 per cent 0 to 7.0 " at least 50 per cent
Coarse aggregate:	0 to 15 mm at least 65 per cent 0 to 30 mm at least 85 per cent 0 to 70 mm at least 100 per cent
Water.....	7 to 8 per cent by weight

The coarsest material (30 to 50 mm) must never constitute more than 15 per cent of the total mix. A 6-in. diameter pipe is customarily used. If material coarser than that given in the foregoing table is used a larger size pipe must be used.

Portland and trass-cements are recommended. Certain types of quick setting cements must be treated with suspicion. The minimum amount of cement is placed at 420 lbs. per cu. yd. for a well-proportioned mix. If the latter is not possible then the amount of cement must be increased above the minimum given. The use of a very fine sand containing clay, lime dust, or quartz-sand up to as high as 2 per cent, may be advisable at times.

One machine (Type L7-Torcrete) operating at a pressure of 6 Atm (limits of delivery—270 yards distance and 45 yards vertically) requires from 30 to 40 H.P. Its approximate average capacity is given as 16 yd. per hour. Energy consumption is said to vary within wide limits under varying conditions.

A number of actual jobs on which this method of placement was used are described giving average figures of placement costs. Figures on pipe deterioration are also given.

Five to six men are required to serve a machine to be kept in continuous operation. The valves must be carefully watched at all times and require considerable attention,

and stoppages should never exceed fifteen minutes. This method is said to offer advantages in intermediate and large scale operations.

Tests of beams reinforced with bundle-bars

HOMER H. HADLEY, *Civil Engineering*, Vol. 11, No. 2, Feb., 1941, pp. 90-93. Reviewed by J. R. SHANK

Four beams 6 in. x 12 in x 10 ft. were made and tested in the laboratories of the City of Seattle. One of these (B1) was reinforced with one straight one-in. sq. bar, another (B2) with the same size bar hooked over the supports, and the other two (B3 and B4), each with four ½-in. sq. bars tightly bundled, two above two. The lower two bars for both were hooked over the supports. The upper two for both were bent up, one at a time for web reinforcement, the one nearest the mid-span was turned back on a standard radius and the other continued along the top of the beam for bond. Beam B4 differed from B3 in the pouring. When 4-in. of it had been poured, the top was roughly leveled off and covered with heavy brown paper. The pouring was then continued without punching holes through this film of paper. The two one-in. sq. bar beams contained ⅜-in. round stirrups spaced 8-in. in the regions of shear. The effective depth for all was 10½-in. The concrete was poured with a 3 to 4-in. slump and withstood 4530 p.s.i. at 14 days. The steel was intermediate-grade new billet stock.

Loading was at two points 12-in. apart on a span of 8-ft. The ultimate loads were, in order from B1 to B4: 22,550, 23,010, 23,980, and 24,660 pounds. All beams failed in the steel and at mid-span. No shear cracks were in evidence and there was no sign of horizontal shear failure at the paper film of B4. The ⅛-in. openings between the ½-in. bars were completely filled with mortar.

The two ends of all beams were further tested, the one-in. sq. bar beams (B1 and B2) with central loading on 30-in. spans and the others (B3 and B4) on 20-in. spans. In all cases one point of support was identical with that of the first test. The ultimate loads, average of two ends, for these tests were, in order: 35,430, 54,675, 63,050, and 73,910 lbs. The 20-in. span for beams B3 and B4 was selected so as to get the bent up bar directly under the central load to eliminate most of its web reinforcement effect. The test of one end of B3 showed 81,630 lbs. ultimated load. The author cannot explain this very high value and prefers to eliminate it. That of the other end was 66,190 lbs. In all cases failure came with the formation of a diagonal crack on the side adjacent to the end and not toward the middle of the beam, although it was on the inner side that the bars had been most highly stressed in the original testing. There was no slippage or trace of movement in the B4 specimens as a result of the paper diaphragm.

These tests indicated that failure to get the usually specified distance between bars is not a serious condition when good placement conditions are obtained and that the bond of concrete to a wet brown paper film is sufficient to provide enough horizontal shearing resistance so that failure by horizontal shear in rather extreme cases of external shear is obviated.

Fundamentals for the practice of prestressed concrete

R. OPPERMANN, *Beton und Eisen*, Vol. 39, No. 11, p. 141 (June 5, 1940) Reviewed by A. U. TEUER

This article describes in detail tests made on two large-sized prestressed reinforced concrete beams to provide a more reliable guide for design. The first of the two beams was constructed in the field as a model and in the ratio of 1:3 to beams of the type now in service. The second beam, similar to the first, was constructed

and tested in a laboratory having temperature and humidity control. The theory and practice of prestressed concrete having been treated in detail in Beton u. Eisen, Vol. 10 (1937), no further consideration is given to these phases.

The two beams tested differed slightly in dimensions, but the following average figures will perhaps serve. They were both I-shaped with a flange width of 9.2 in., total depth 46.5 in., span 60 ft., and a web thickness varying from 3.15 to 1.57 in.

The main horizontal reinforcement consisted of 64 – 0.21-in. diameter plain bars of special steel and carried a pre-stress of 78,400 p.s.i. Some additional horizontal steel in the upper flange and web were not prestressed. The stirrups were placed 15 in. on center and carried a prestress of 85,500 p.s.i. The design was for a combined dead and live load of 825 lb. per running ft. Certain other design data, as well as a description of the materials, methods of manufacture, prestressing of the steel and time schedules are given. The beams were loaded by means of four hydraulic presses bearing on twelve uniformly spaced points through the medium of levers. Deflection measurements were taken both at the mid-and the quarter-points and certain deformation readings made, all to a precision of .00003 in.

Since the beams showed approximately the same behavior in all essential details, only that of the second (Dresden), subject to the more extensive measurements, will be described. This beam was loaded up to its maximum design load three times at intervals of a month. In between loadings the beam was kept under a constant load two-thirds that of the design load. Direct proportionality between load and deflection existed up to a moment of 532,000 ft.-lb. In the third cycle the loading was carried up to the ultimate or 905,000 ft-lb. The first visible crack appeared at 576,000 ft-lb. Failure followed by crushing of the concrete. The last reliable deflection measurement was 4.05 in.

The maximum crack width was less than .04 in. and a rather uniform spacing of cracks was noted during test. After failure, all the cracks in the tension area closed, but a new set in the compressive area appeared. This is taken to demonstrate clearly the continued existence of prestress in the two halves of the ruptured beam, and the dependability of the bond action.

Dependent upon these tests the following deductions are made: The modulus of elasticity on the basis of a deflection of 0.72 in. was 4,520,000 p.s.i. at 3 weeks and 5,300,000 p.s.i. at 4 months. A value of 7 is hence placed on n for design. Since great similarity and uniformity were observed in the deformations of the two beams within the range of design loads, an overload of 1.5 can still be considered to lie within the usual limits of safe practice. On the assumption of an average concrete tensile stress of 500 p.s.i., and the fact that the first tension cracks, even if not visible, probably occur at the limit of proportionality, observed at 535,000 ft-lb., an effective pre-stress of 64,000 p.s.i. may be used for design. The crack diagrams at high loads yield a clear picture of the stress conditions. Cracks due to localized overstressing were not apparent at any time, and bar anchorages used may be taken as adequate. The following computed stress values at beam failure set against the ultimate stress in materials tests are tabulated (see next page).

To be emphasized here is the fact that failure of both beams took place as the computed values would indicate; beam No. 1 by crushing of the concrete, beam No. 2 by tension in the steel.

An additional deduction from these tests is that an assumed loss of 21,000 p.s.i. in prestress due to shrinkage and creep is quite adequate in view of the actual loss of 9,900 p.s.i. measured.

	Computed Beam Stresses	Materials Ultimate Stress
Beam No. 1		
Concrete stress, p.s.i.....	5,410	5,860
Steel stress, p.s.i.....	117,000	147,000
Beam No. 2		
Concrete stress, p.s.i.....	6,400	7,150
Steel stress, p.s.i.....	131,000	131,000

Prestressing of the stirrups is considered necessary to assure an adequate shear strength. That such shear strength was attained is illustrated in these tests by failure in flexure at the beam centers. A prestress of 57,000 p.s.i. is justified in design for steels having a tensile strength of 142,000 p.s.i. For design the concrete stress may be taken as 0.35 cube strength.

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 37—1941

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 12, Sept. 1940, to June 1941 and Supplement, November 1941

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference is by supplementary page numbers. If the last page of a paper is 28, the first page of the discussion, published later, is 28-1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Housing," and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

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